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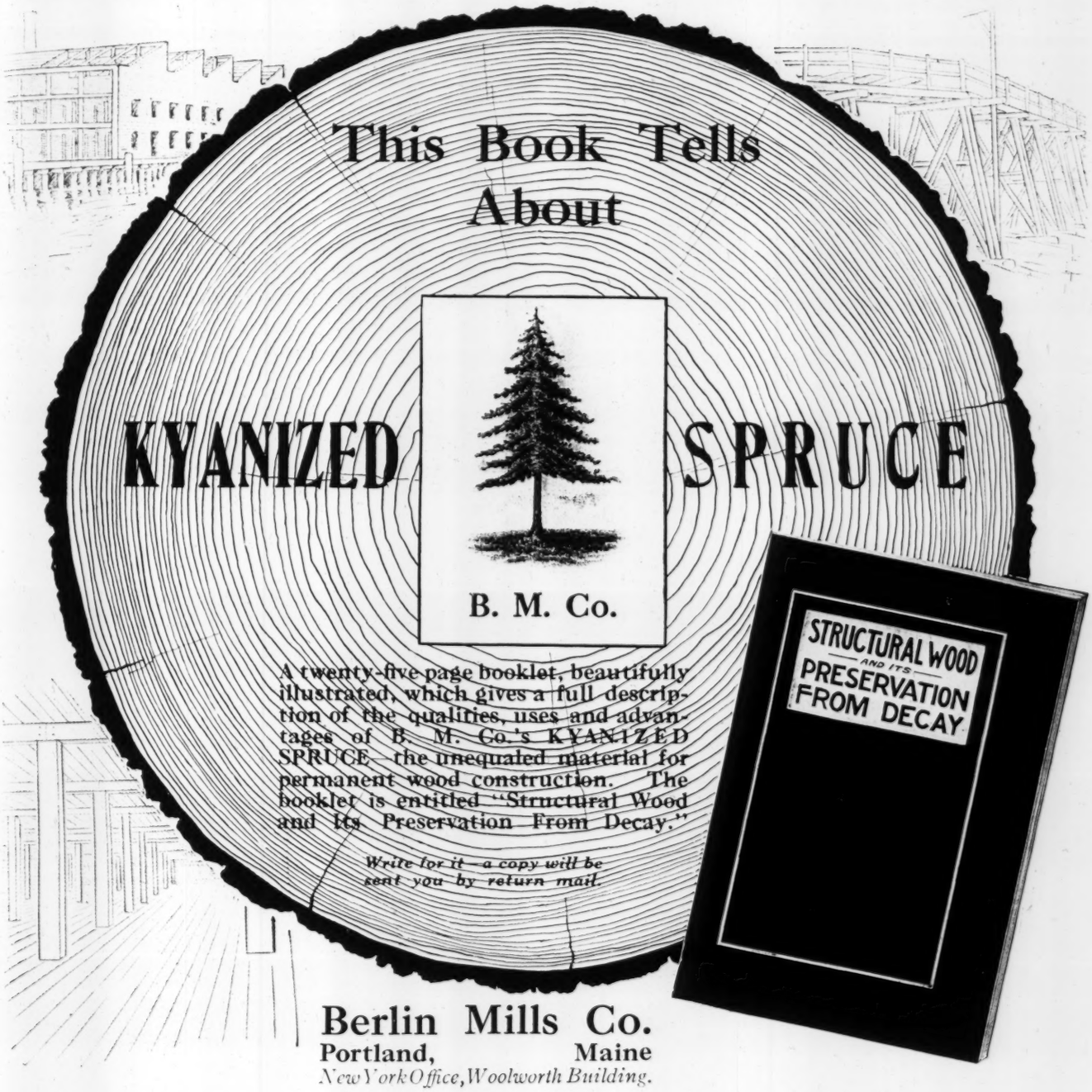
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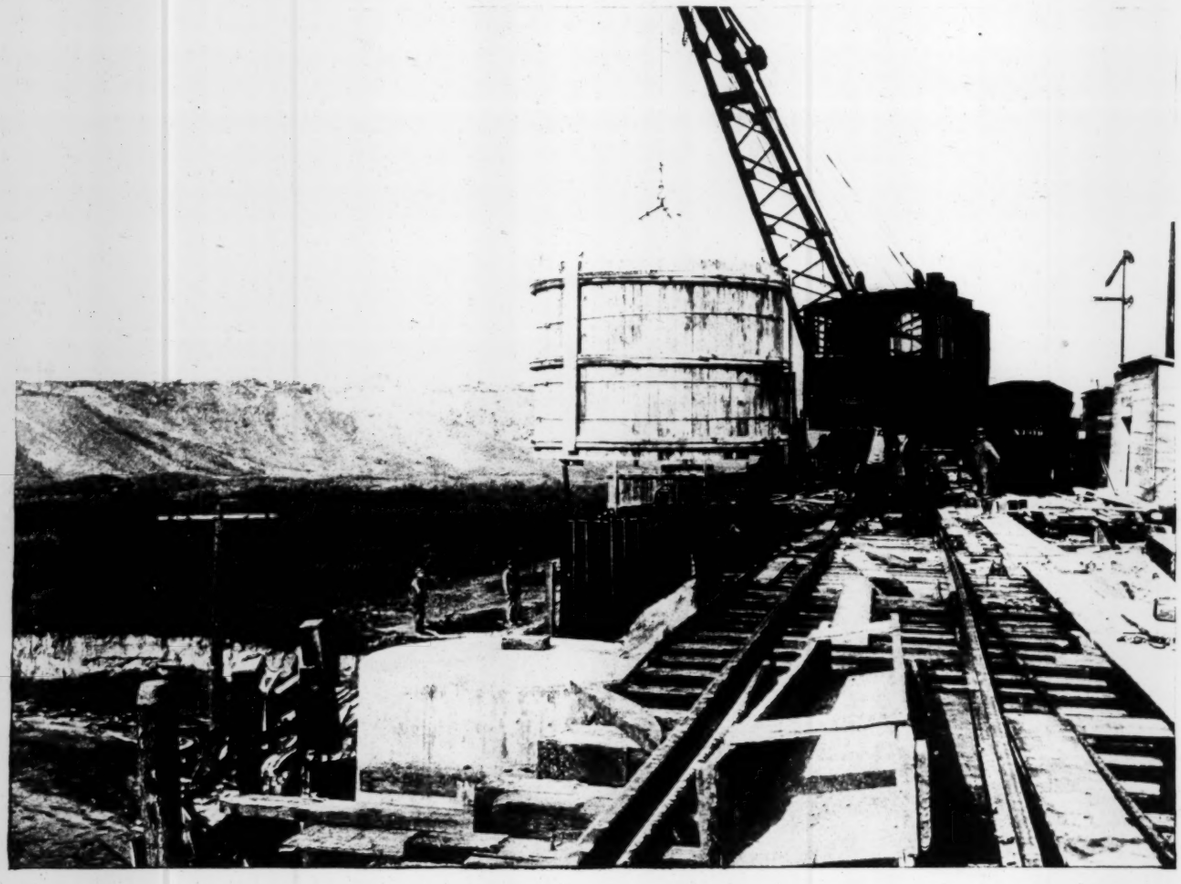

The Lake Shore Bridge over Buffalo River

VI. Removing Forms from Complete Piers

This picture shows, on the left, the top of the north completed concrete pier from which the wooden form is being lifted by the traveling crane. The wooden forms for the tops of the cylinders resembled water tanks without bottoms, being composed of long staves bound together by three hoops.

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Engineering News

CHARLES WHITING BAKER, *Editor*

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What The Buyer Needs To Know

Number Thirty-One

Here is a statement recently issued by the Wichita (Kans.) "Beacon" which illustrates how the demand for *truthful* advertising is being met by publishers throughout the country.

"The 'Beacon' is going to make an effort hereafter to be responsible for the character of advertising which appears in its columns. The paper will endeavor to educate the public to believe that what it sees in the news columns and in the advertising columns is alike worthy of serious attention as truthful statements."

From the fake mining schemes and doped patent medicine advertisements of ten years ago to the above is a far cry.

But publishers and advertisers have realized for some time that advertisements must tell the truth if the business of both is to prosper.

For the success of any business depends upon the confidence the public has in it.

Confidence begot of the truth found in its advertisements.

And to so great a degree is this true that a single untruthful statement contained in an advertisement may be the little pill with which the Advertiser commits Business Suicide.

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Just three men responded to the ad—and one didn't buy.

Some of his customers who had seen the ad wanted to know whether the business had changed hands, because up to that time they had thought the advertisements of this concern reliable.

We think that the publishers of the paper in which this advertisement appeared are as much to be censured as the merchant who tried to "put one over" on the buyer.

We realized long ago that in admitting a fraudulent advertisement in its columns a publication is fouling its own nest.

That possible purchasers come to regard its advertising columns with suspicion because some of the advertisements contained therein are untrustworthy.

We went further. Since the subscriber of this publication buys it because he knows it helps him in a business or professional way, we decided that no product that could not be used toward this end should be admitted in the Selling Section of Engineering News.

That perfumes, crackers and steam yachts, being products which have no place in the solving of Engineering problems, are debarred from our advertising columns because they don't "belong."

And this has resulted in the formulating of the Hill Advertising Code which, briefly, declares that:

No advertisement can appear in the Engineering News which contains misstatements gross exaggerations, "knocking" copy, fake "free" offers, products not ready to be marketed, or behind which the necessary finances to advertise it profitably are lacking; and products which cannot be used in a business or professional way by the reader.

All of which assures you of the reliability of the manufacturers whose products, truthfully advertised, you find in the Selling Section of Engineering News.

Engineering News

VOLUME 70 JULY 31, 1913 NUMBER 5

The Port of Hamburg*

By I. F. Burendey†

New York and Hamburg are the great commercial metropolises of the American and of the European continent. Their seaports, however, have taken a very different course of development. While in America wide rivers have been available, affording ample berthing space for vessels, the modern development of the port of Hamburg has necessitated a digging of new basins out of the ground. The necessity for such a mode of procedure results from the fact that the Elbe River is only 860 ft. wide between the harbor works on both sides, whereas there is a sheer width of 2100 ft. between the pierheads of the Hudson.

*From an address before the American Society of Mechanical Engineers during its visit to Hamburg, June 24.

†Director of the Port of Hamburg, Germany.

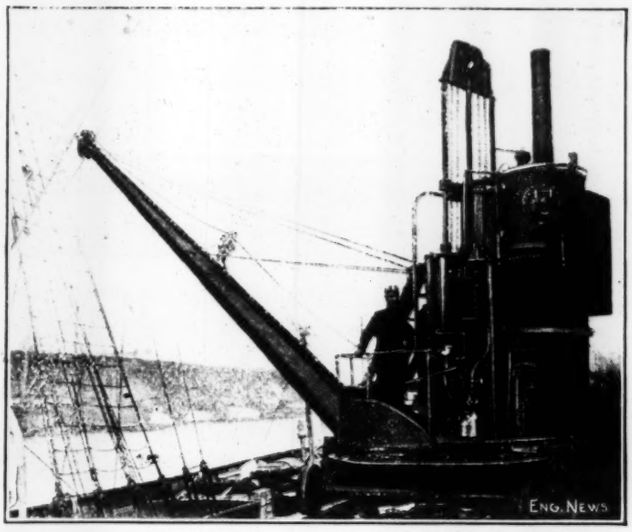


Fig. 1. Old Type of Steam-Operated Traveling Jib Crane Still Used in the Port of Hamburg

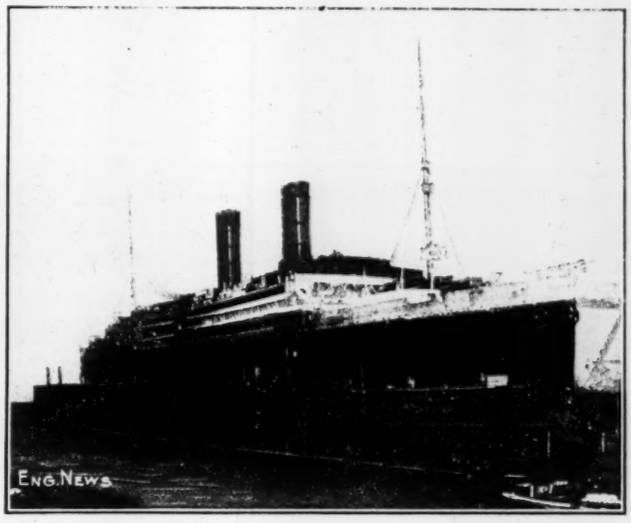


Fig. 3. The "Imperator" on the Floating Dry Dock at the Blohm & Voss Shipyards

Fig. 2. 250-Ton Crane at Blohm & Voss Shipyards

Large basins have therefore been excavated at Hamburg during the past fifty years on places which used to be pasture ground for cattle.

The chief part of the harbor itself is outside of the German Customs limits and constitutes a free port, enabling

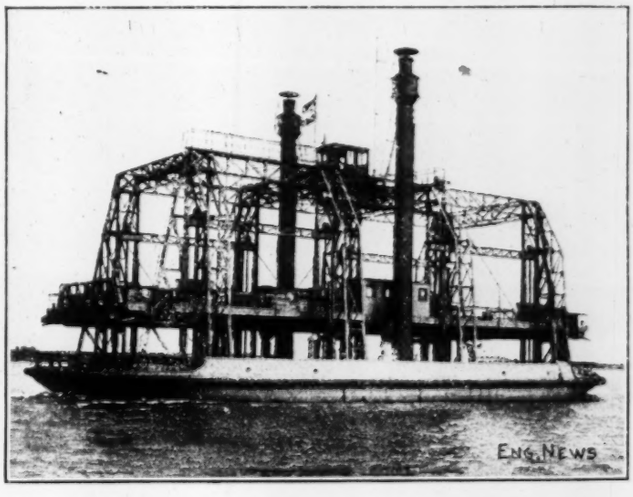


Fig. 4. A German Car Float with Adjustable Dock

the ships to come right up to the city without being examined by the custom-house officers. Duty is not paid until the goods cross the Customs limits on the way to the city, to the inland or to the railway stations.

At present there are available at Hamburg 130,000 lineal feet of quays for ocean liners, more than five million square feet of sheds and 805 cranes for loading and unloading vessels.

As the present harbor facilities have become inadequate to the ever increasing traffic, work was started two years

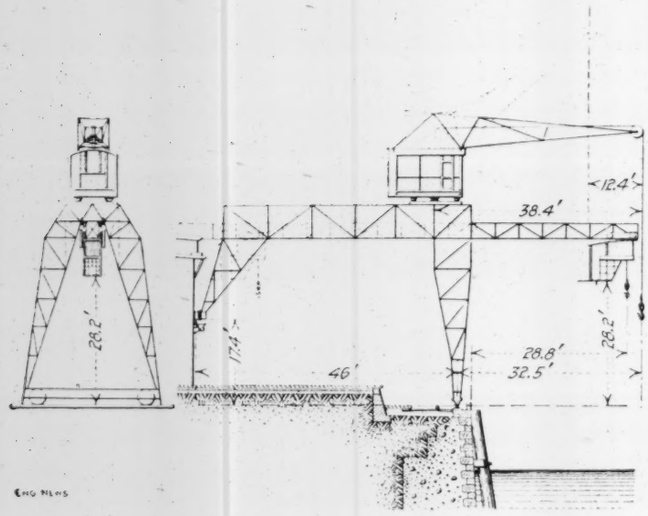


FIG. 5. COMBINED JIB CRANE AND TRAVELING CONVEYOR FOR HAMBURG QUAY WALL

ago to considerably increase the number of the existing basins. The next work to be done at Waltershof calls for an expenditure of 45,100,000 marks. It will eventually be possible to erect more than 40,000 lineal feet of quay walls on this territory.

An idea of the volume of the maritime business of Hamburg may be gained from the fact that in 1912 there arrived at our port, 18,500 vessels, representing more than fourteen million registered tons.

The sheds along the quays of Hamburg are not supposed to store the goods for any considerable length of time. The cargoes taken from the ships are merely assorted here, to be conveyed immediately, either by boat or by rail to the warehouses of the city or inland.

The modern pier sheds are 200 ft. wide in order that all the goods taken from one ship may be distributed opposite to it for further treatment. By the railroad tracks at the rear of the shed the goods are carried away. All sheds are built of wood because steel structures would be far more expensive and in case of fire would be destroyed anyway.

The hoisting of the goods from the ship's hold is done by revolving jib cranes of which there are 650 available. These cranes have an average lifting capacity of 3 tons. The oldest of them, shown in Fig. 1, are operated by steam.

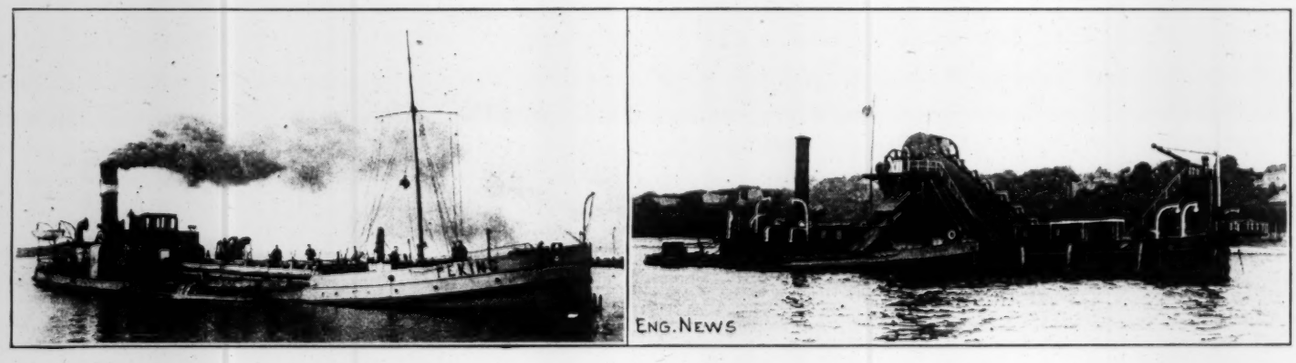
Twenty years ago the first experiments were made with cranes operated by electricity. All cranes constructed since are electrically operated. The feet of these cranes move along the quay on two rails. One is placed close to the edge of the quay while the other one is fixed at some height to the wall of the shed. The crane thus spans the railroad track without obstructing to any great extent the working space on the pier.

The great advantage of these cranes lies in their capability of adapting themselves automatically to any load. Owing to this and to their simplicity they perform work very economically. The lifting of the cargo and the slueing of the crane are done by separate motors, while for the movement along the quay manual labor is employed. There is at least one crane to every 65 ft. of pier. It often happens that two or even three cranes are taking cargo from one hatchway.

Very recently the desire for greater rapidity in loading and discharging, thereby shortening the stay of the vessels at the pier, has led to the construction of double cranes shown in outline in Fig. 5. They consist of a revolving jib crane traveling on top of the pedestal and a lower flange conveyor suspended from the pedestal. The jib crane will transfer goods of any kind and any volume from the ship's deck to the shed floor, while the capacity of the conveyor is limited to goods of lesser bulk, as sacks, parcels, bales, small boxes, and so on, the clear width between the legs of the pedestal being a limited one.

Fig. 2 shows the 250-ton crane at the shipyard of Messrs. Blohm & Voss, which has just been completed. This is the largest crane in the world. At a radius of 164 ft. the crane will lift a load of 10 tons and at 105 ft. will lift 250 tons.

From the sheds on the group the goods are distributed to warehouses either by lighter or by truck. There is a large number of such warehouses in the Free Port district; they are built alongside canals which are of sufficient depth for river barges. These warehouses occupy



FIGS. 6 AND 7. TYPICAL SUCTION AND LADDER DREDGES AT WORK ON THE ELBE CHANNEL

a land area of more than a million square feet. The port of Hamburg is 65 nautical miles above the mouth of the Elbe River. Sixty years ago the channel of the Elbe had a depth of only 15 ft. at high tide. By steady dredging this depth has been increased at present to 37 ft. and will be deepened to at least 40 ft. in a few years. Mean low water at Hamburg is 6½ ft. below mean high water, and at the mouth of the Elbe the difference is 9 ft. At the present time there are 16 state-owned dredges and 6 chartered dredges at work on the Elbe's channel. In recent years 8 million cu.yd. on an average have been dredged annually.

The dredged material is used for the filling of lowlands along the river. By means of pumps it is carried through a pipe line a long distance over land.

Since 1877 the Elbe from the North Sea to Hamburg has been kept open for navigation even in the coldest winters. The ice is broken up by stanch steamers equipped with a prow which enables them to shove themselves upon the ice, breaking it by their weight.

Fig. 4 shows a car float by which railway cars are transferred across a branch of the Elbe which must not be bridged. The car tracks are carried upon a platform mounted on screws. By means of these screws the position of the platform may be changed according to the height of the tide.

In order to improve the trucking facilities from the city to the docks on the other side of the river and for the benefit of the workmen employed by the shipbuilding yards two tunnels 20 ft. in diameter have been built under the Elbe. One is for the outbound, the other for the inbound traffic. The tunnel has no incline, because the traffic involved is chiefly from waterfront to waterfront. Vehicles and pedestrians are lifted and lowered by electric elevators of 10 tons capacity, the total lift being 80 ft. (See Figs. 8-9.)

During the rush hours trucking through the tubes is suspended, enabling pedestrians to use the tube for its entire width.

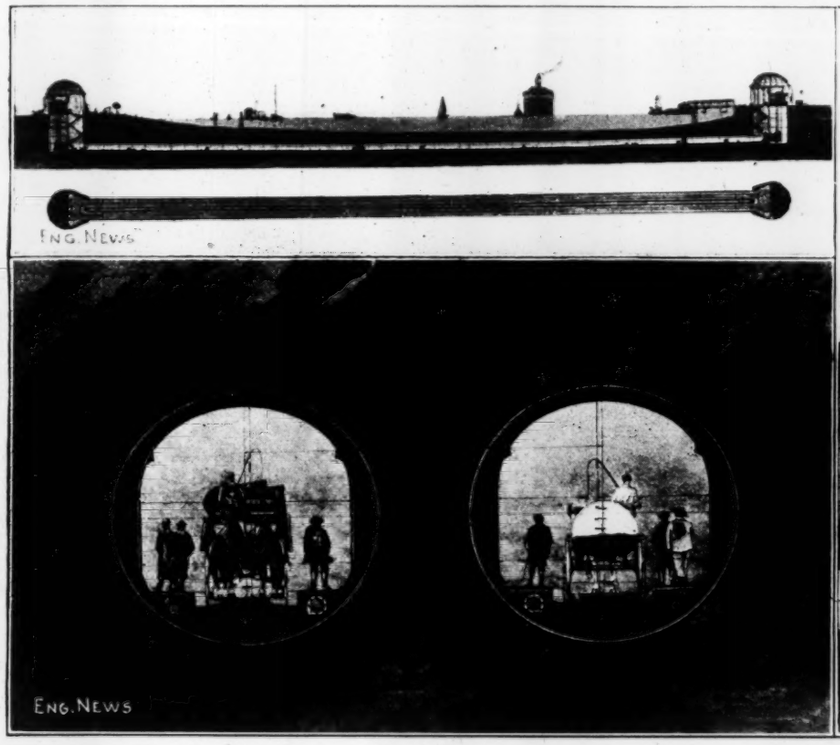


FIG. 8. PLAN, PROFILE AND CROSS-SECTIONS OF TUBE TUNNELS UNDER THE ELBE AT HAMBURG

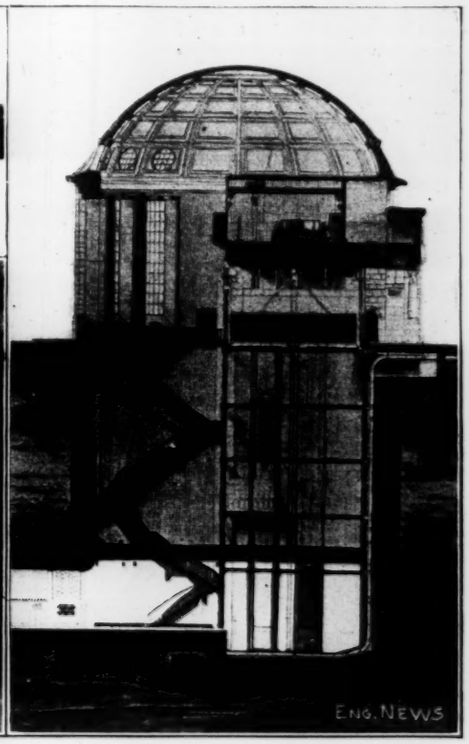


FIG. 9. SECTION OF TERMINAL SHOPS AND ELEVATORS OF ELBE TUNNEL

The Effect of Forest Fires on Standing Hardwood Timber is the title of circular 216, by H. W. Long, Forest Pathologist, Bureau of Plant Industry, dated March 31, 1913, issued by the Department of Agriculture, Washington, D. C. Its aim is to point out the injurious effects of even light fires in a forest.

The most noticeable of these effects are the killing of the young growth, the destruction of the layer of leaves and mulch which protects the soil from washing under heavy rains, and the fire scars left on the trees. The conclusions of the circular are based upon a study made in the Ozark Mountains of Arkansas.

Damage by insects and rot was found in connection with practically every fire scar on the bases of the trees. The fire may have occurred 50 to 100 yr. ago and all signs of it hidden by later growth, but during the two or three years that it took the tree to heal the scar, worms and rot-producing spores entered at the open wound.

The following is quoted from the circular:

The loss of good merchantable timber in a stave operation, from rot and worms alone, is enormous. It is often claimed that light fires do not injure the forest, and even that it is a benefit to burn the forests annually. On one stave sale area in the Ozarks, 76 trees out of every 100 felled had butt rot, and 27 trees in every 100 had wormholes of some kind in them. Not only was there a money loss from the cull of 76 trees, but the expense of felling unsound trees must be considered. For five widely separated areas in the eastern part of this forest an average of 65 trees in every 100 had butt rot, and 26 had wormholes sufficient to cull some of the bolts.

Another way that fires injure standing timber is by burning through sound bark and sapwood into the hollow butts. Once the hollowed-out interior of the tree is exposed to the air, the rot which originally caused the hollow seems to grow up the tree more rapidly, while other fires that follow gradually widen the hollow and destroy the bark at the base of the tree until the tree is killed.

The Barge Canal Crossing of Oak Orchard Creek, Medina, N.Y.

By Noble E. Whitford*

The crossing of Oak Orchard Creek in the village of Medina constitutes the last contract of any considerable size to be awarded for the construction of the New York State Barge Canal. Because of difficulties involved in deciding problems of construction, this short stretch of canal is one of the most interesting portions of the whole line and it has received a greater amount of study than any other section. Plans for this and adjoining portions were begun early in canal operations, and, while this stretch at Medina has been under consideration, construction work on both sides of the gap has been entirely completed.

In following the old canal alignment, which the Barge Canal does throughout a large part of the western section, it encounters at Medina a sharp horseshoe curve, which the builders of the original canal employed, so as to pass around the head of a wide and deep gorge, and which the later builders have continued to follow. Just below the present canal crossing, Oak Orchard Creek passes over a steep declivity and the valley widens into a gorge, which is 500 ft. wide at the top and 90 ft. deep, at the point where a direct line would cross it in connecting the straight stretches of canal on either side.

Proposed Schemes for Crossing

To carry the canal across this gorge secures excellent alignment, but involves serious difficulties. However, all of the early studies contemplated some method of crossing at this point. To follow the alignment of the old canal, which is the plan finally adopted, also has its serious problems (Fig. 1).

Soon after the canal was authorized two tentative schemes of crossing the gorge were considered. The first was by means of an earth fill, on which was to be a channel of standard embankment section with the usual inner and outer bank slopes. This plan gave an extremely wide fill and was not very seriously considered. The other was by means of a steel trough supported by steel trestle work, resting on concrete piers. Plans along these lines were not worked out in so careful detail as during the later studies.

When, after a little, the consideration of this locality was begun in earnest, complete plans and estimates were made for seven schemes. A comparison of these seven methods of accomplishing the same result becomes interesting, since it shows the advantages of a concrete structure over a steel structure, in both first cost and maintenance charges, as well as in permanence, and it shows also that both of these structures may be made.

The seven schemes and their costs, which include simply the length of canal across the gorge, are as follows:

(1) A steel cantilever structure with a steel trough for the canal, which was estimated to cost $500,000.

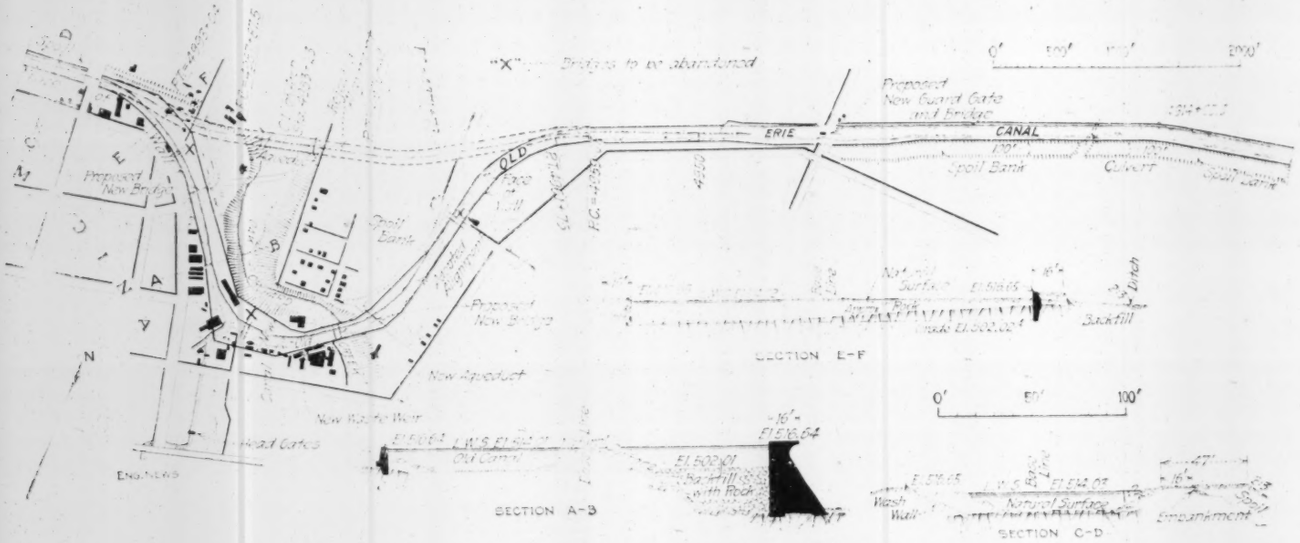


Fig. 1. Plan and Typical Sections of Adopted Crossing of New York State Barge Canal over Old Orchard Creek, Medina, N. Y.

(Dotted lines show originally proposed short alignment with 300-ft. arch.)

(2) A three-hinged steel arch of 300 ft. span, c. to c., carrying a steel trough for its length of 300 ft., the approaches of about a hundred feet on either side being concrete troughs supported on short concrete arches. The estimated cost of this structure was $405,000.

(3) The same steel arch and trough, but with approaches built all of steel, the cost being $407,000.

(4) A concrete arch in place of the steel arch, the remainder of the structure, both trough and approach supports, being of steel, at an estimated cost of $300,000.

(5) An all-concrete structure, consisting of a plain concrete trough on a concrete arch of 300-ft. span, c. to c., which was estimated at $272,000.

(6) This concrete structure, but with architectural adornment, which would raise the cost to $285,000.

(7) An earth fill over a long culvert, carrying the canal in a concrete trough.

After a study of these seven schemes has been made and the costs and advantages of each had been carefully considered, No. 6 was adopted for making complete detail plans, on which to let the contract. The reasons for this choice, stated briefly, were that this concrete structure was considered to be the safest, the most permanent,

*Resident Engineer, Department of State Engineer and Surveyor, Albany, N. Y.

the cheapest to build, the cheapest to maintain and the most pleasing in appearance. Complete contract drawings were made along the adopted scheme and for some time it seemed that the canal would be built in accordance with these plans.

The question was often asked, especially by those not connected with the engineering force, as to why an earth fill was not the simplest and best solution of the problem. The considerably greater cost is one answer, and another is indicated in the experience of the past year at a place where the new canal has been built upon a heavy embankment—at the Irondequoit Creek crossing, a few miles east of Rochester. Here a very serious break occurred last summer and it will be necessary to rebuild.* The form of construction to be used in this work of rebuilding is still under consideration.

In making detail plans for the Oak Orchard Creek crossing the arch was slightly shortened over the tentative scheme. The perfected plans called for a single-span concrete arch, which, if built, would have had a clear span of 285 ft., a total width of structure of 139 ft. and a total length of 508 ft. It would have had a trunk of 103 ft. clear width for carrying the canal prism, in which there would have been a maximum depth of 13 ft. 2 in. of water, weighing 825 lb. per sq.ft. The total weight of water to be carried by the single span would have been 12,100 tons, while the total load, including the weight of the structure, would have been 46,000 tons.

By reason of its width and length of span, the great load to be carried and the large cost, this structure became, probably, the most important piece of concrete arch construction that had ever been planned up to that time. It was not deemed wise, therefore, to attempt its building until additional information had been acquired concerning the behavior of concrete under certain conditions of stress. Accordingly, at the expense of several thousand dollars, tests were made on concrete cubes and prisms and on a dozen models of the arch itself. An account of these tests was printed in ENGINEERING NEWS† shortly after they were made.

The model arches were all stressed to destruction and showed an ultimate strength of ten or twelve times the proportionate load which the large structure would have been called upon to carry. Also they disproved what some had feared—that the failure would be by shear at the springing—although such failure was encouraged in several of the models by making notches that greatly reduced the sectional area at this point.

The advantages of crossing the gorge rather than making the detour around its head were several and important and the plan was not abandoned without serious consideration. In the first place, a nearly straight alignment could be obtained, while the curves by the old route were decidedly objectionable. Also the new alignment was about a half-mile shorter. From the standpoint of the navigator these were two very desirable advantages. Moreover, the direct line would eliminate several fixed bridges and would require no lift bridge. It would obviate the necessity of taking much valuable property, which the other route, passing through the center of Medina, cannot avoid. Furthermore, the cost would be considerably less.

The big arch would have been partly experimental, but perhaps the chief consideration in deciding against it was the nature of the foundation, which is a soft sandstone, known locally as "red horse." The abutments were spread with a view to obtaining a perfectly stable foundation even on this soft rock, but this element of uncertainty and the serious results that might follow a settlement caused a final rejection of the plan.

Before the present plans were undertaken, another scheme for crossing the gorge was tried. This time it was by means of a rock fill. In the vicinity of Medina quarries have been operated for many years and there has accumulated a large quantity of spalls. It was ascertained that these could be obtained cheaply and the scheme contemplated their use for the fill. On top there was to be a cushion of earth and then a concrete trough, the sides being backed by small earth embankments. The estimates showed this project to be about $90,000 cheaper than the concrete arch, but detail plans were never worked out.

THE ADOPTED DESIGN

Although the long arch has been eliminated, the contract as awarded (known as Contract No. 65) has several interesting features and also a variety of construction that includes almost all important kinds on the canal, with the exception of locks. It has about two miles of excavation for canal prism, through both earth and rock. It has embankments and wash wall, thousands of feet of retaining walls, dock walls, concrete culverts, highway construction, several fixed and lift bridges with their approaches, a guard-gate, a waste-weir and sluice gates. To cross the stream a concrete arch of 50-ft. span will be built. At certain points the new canal bottom will be above the natural surface of the ground. Here the bed of the prism will be a rock fill, while the downstream side will be a wall resting upon the natural rock, which in effect will be a long dam, sustaining hydrostatic pressure for its full height, which reaches a maximum of 45 ft. (Fig. 2).

The old canal in this vicinity is somewhat wider than the standard section, which had a width at water surface of 70 ft. To facilitate feeding, the width on the 60-mile Lockport-Rochester level tapered from a 99-ft. section at Lockport to the standard width at Rochester. The standard depth of 7 ft. was also increased in the western portion. At certain points the depth of the old canal was increased by two feet during the enlargement of 1896-8, but until the present improvement is completed, 7 ft. continues to be the limiting depth. In the eastern portion of this contract the new canal will have the usual bottom width for excavation sections—75 ft. The depth, 12 ft., is that which obtains throughout the canal, except where natural deep water is encountered. In this stretch there will be some sections in rock cut, the standard width for this section being 94 ft., which is somewhat increased at curves.

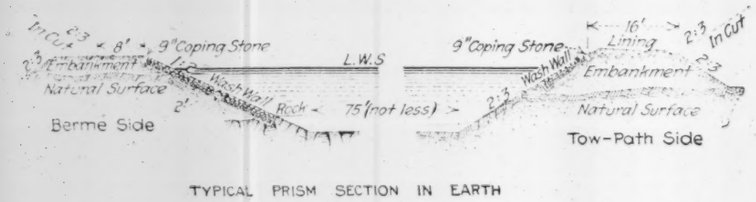
In making the detour through Medina the old canal has considerable sharp curvature, the sharpest being a curve of 10°. A device to relieve this sharp curvature in the new work is obtained by making the channel much wider than usual. An extreme width of 350 ft. is provided at one point. This width, however, is partly for

*"Engineering News," Sept. 19, 1912, p. 548; Oct. 3, 1912, p. 622.

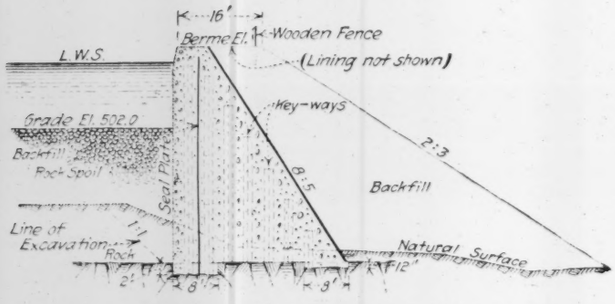
†March 10, 1910, p. 274, 282; July 7, 1910, p. 21.

another purpose, since it also serves to form a harbor in the village.

Not only at these sharp curves is the channel widened, but at all places where the prism is confined by embankment or wall the bottom width has been made 110 ft. The minimum width specified in the Barge Canal law is 75 ft. In embankment section the width can be increased with little or no extra expense and it has been the general practice throughout the canal to make the bottom width 110 ft. under these conditions. This precaution has been taken because it seemed wise to make this provision against a future possible widening, since it could be done without additional cost. By amendment to the Barge Canal law the locks were increased from 28 to 45 ft. wide, but the channel width was not changed. Thus, it came about that the locks are capable of passing boats of so great width that two of them cannot meet in the minimum channel. Another precaution has also been taken; to avoid the necessity of rehandling excavated material, should this widening ever take place, the engineers have required all spoil banks to be set back a sufficient distance from the excavated prism.



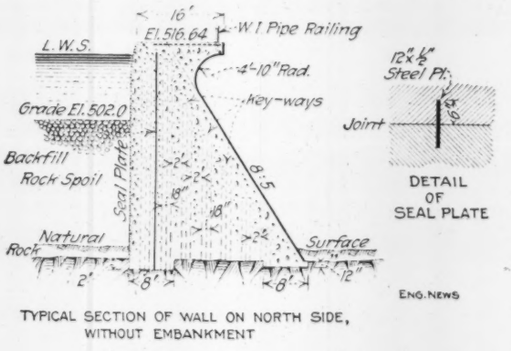
TYPICAL PRISM SECTION IN EARTH



TYPICAL SECTION OF WALL ON NORTH SIDE WHERE BACKED BY EMBANKMENT

RETAINING WALL—The amount of retaining wall on so short a contract as that at Medina is rather unusual and some of this wall is so high as to become interesting. On the north side of the canal there is planned a continuous stretch of nearly 7000 lin.ft., while on the south side two pieces will measure together about 4600 ft. long. Between fifteen and sixteen hundred feet of the north wall will be high, extending at some places far below canal bottom to the low ground as it slopes down toward the gorge. Thus, as has been said, the canal in effect will be running behind a dam for nearly a third of a mile. This high wall has a gravity section sufficient to resist the water pressure. (Fig. 2.) Where the natural surface does not drop off too abruptly, an embankment will be placed outside the wall, which not only will add to its strength, but will give a 16-ft. top for a roadway. Where the wall stands without embankment, including the north side of the aqueduct, this same width of top is secured by extending the concrete in an overhang, as shown in the sketch (Fig. 2). As a preventive against leakage at the joints between construction sections a vertical plate of steel, called a seal plate, is embedded 6 in. in each of the adjoining sections. The same device is used at the joints in the acqueduct and a more detailed description is given below in the discussion of that structure. In addition to the seal plate the customary keyways are to be used in securing a bond between adjacent sections.

GUARD GATE—One of the structures on this contract is a guard-gate. On the long level between Rochester and Lockport it has been the practice to place these structures at intervals of not more than ten miles. The one near Medina is typical of most of the double gates on the canal. It has two openings of 55 ft. each, with a central pier of 9-ft. width between. The placing of this gate on abutments which are had in common with a highway bridge suggests a noteworthy practice in planning Barge Canal bridges. It has been customary to place bridge abutments at least 110 ft. apart, measured at right angles to the axis of the canal. This is a precautionary



TYPICAL SECTION OF WALL ON NORTH SIDE, WITHOUT EMBANKMENT

FIG. 2. ENLARGED TYPICAL SECTIONS THROUGH CANAL

measure against possible widening. To be of use in a 110-ft. channel the abutments that are only 110 ft. apart must be carried below canal bottom. To obviate this necessity, many of the abutments have been set back and the bridge span lengthened. By locating the guard-gate and bridge together there is economy in concrete and no necessity of placing the abutments farther apart than is required for the gate.

AQUEDUCT—While the aqueduct (Figs. 3-4) will not be so spectacular as the long one first planned, still it will be an interesting structure. Since breaches in canal prisms generally occur where the channel crosses a stream by means of culvert or other structure, the Medina aqueduct has been designed with the idea of making it practically failure proof. The stresses in the concrete of the arch ring are very low.

The aqueduct is to be founded on the well known Medina sandstone, which is much harder than the rock encountered in the bottom of the gorge and is considered an entirely safe foundation. The side walls are of the solid gravity type, designed to withstand not only the water pressure, but also any blow they may receive from loaded barges. The arch ring is made especially strong and steel ribs are embedded as an extra factor of safety.

The arch is to be built in five sections of 25-ft. width each (Fig. 4). The whole arch is to be waterproofed after all the sections are laid, but as a special precaution against leaking at the sectional joints the plans call for a 12-in. steel plate of ½-in. metal to be embedded for 6 in. of its width at the face of each alternate arch ring. When the adjacent ring is laid, the projecting portion

of the plate becomes embedded in its concrete. There is a single plate in the center of the ring at the joints of the arch sections, but two plates, each 6 in. from the center, are used where the arch joins the side walls. The arch seal plates lap by plates embedded in the abutments and extending above the skewback. The contractor will be required to clean very thoroughly the half of the seal plate that is to be embedded at the first operation, so as to insure perfect adhesion, and at the same time he will be required to grease the other half, so as to prevent adhesion till the adjoining section is to be laid.

One difficulty in prosecuting the work on this contract arises from the necessity of maintaining navigation during the open season. At the aqueduct this requirement will cause the most serious trouble, for it means the tearing out of the old aqueduct and the building of the new in one winter season—between November 15 and May 15. One clause of the specifications reads:

> In order to protect the concrete in the arch from freezing the contractor will be required to enclose the work in a suitable shed, either fixed or movable, heated to the necessary temperature, or at the option of the State Engineer the contractor shall provide forms so designed that the concrete can be heated. The removal of existing aqueduct and work on the new aqueduct, to be built under this contract, must be conducted in such a manner as not to interfere with the navigation of the canal.

The clear span of this aqueduct is 50 ft. The arch ring has a thickness of 3 ft. at the crown and of 6 ft. 10½ in. at the springing line. The rise is 12 ft., the radius of the intrados, 32 ft., and that of the extrados, 49 ft. The clear width of the channel between side walls is 125 ft. Five 25-ft. sections of arch make up this width.

Each of these sections contains four reinforcing ribs. (Fig. 4.) Each rib is a box, having 3x3-in. angles at the four corners with diagonal lacing bars on top and bottom and both sides. The four ribs will be held together by eleven laterals of 2½x2½-in. angles—on top and botton. where the splices are made with the portions embedded in the abutments; on top at the center; at two other intermediate places, both top and bottom; and on top at the extreme ends, within the abutments. Three anchor clips are fastened near the ends of each of the four angles that make up each rib, on the top portions within the abutments.

Especial care will be taken to prevent leakage. The arch rings will be waterproofed and the foundations of the side walls, and of the high walls in general, will be carefully prepared. The rock surface must be scrupulously freed from loose fragments and, if required, seams shall be followed by a drill, and then cleaned and filled with grout under pressure.

For waterproofing the arch the specifications make the following requirements: The concrete surface shall be coated with hot coal-tar pitch, allowing 5 gal. per 100 sq.ft. Upon this coating, while hot, shall be laid one layer of two-ply, double-thickness, coal-tar felt, weighing not less than 28 lb. per 100 sq.ft. A second coat of pitch and another layer of felt shall be laid immediately in like manner. Upon the third coat of pitch a layer of one-ply felt reinforced with burlap must be laid. After another application of pitch, two layers of two-ply, double-thickness felt shall be laid, and then upon another coat of pitch a single thickness of hard-burned, cherry-red brick shall be laid in cement. The pitch must be of best quality of straight run coal-tar pitch, distilled direct from American coal-tar, and must have a melting point of about 140° F. All sheets of felt must have laps of 10

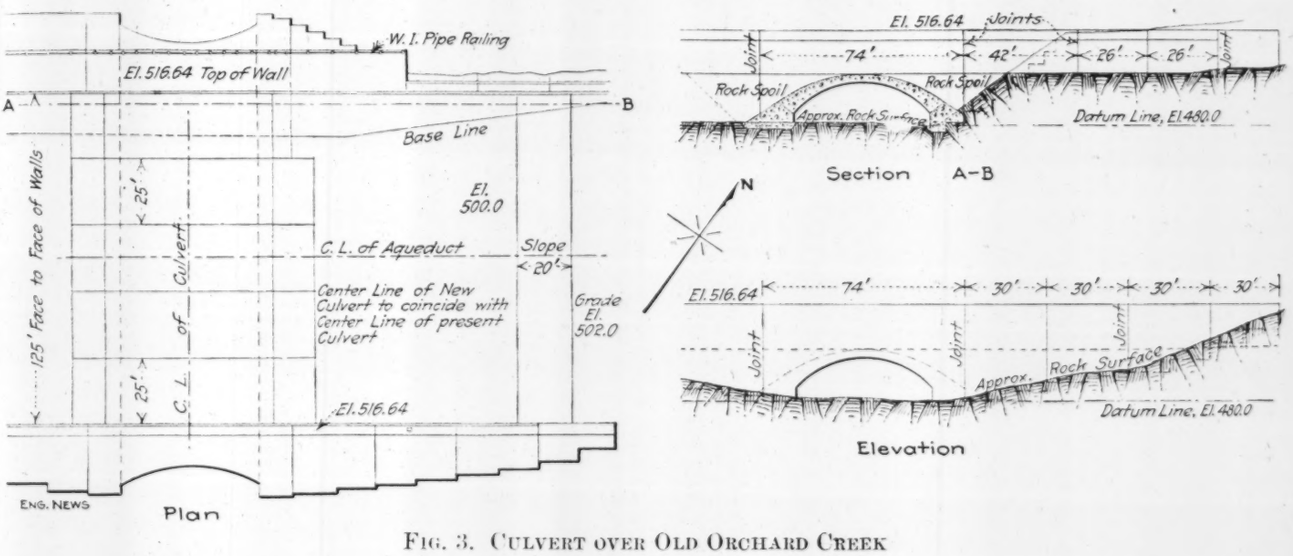


FIG. 3. CULVERT OVER OLD ORCHARD CREEK

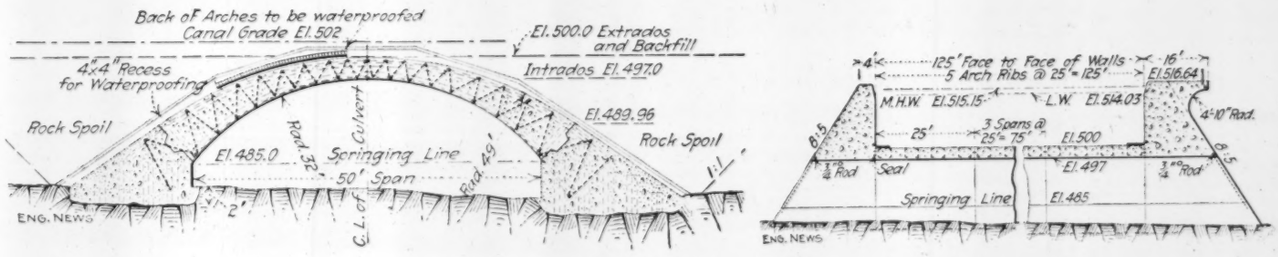


FIG. 4. DETAILS OF CULVERT DESIGN

in., and the laying must start at the lowest point of the skewbacks and extend transversely to the arch axis.

A noteworthy device has been employed to prevent the waterproofing material from cleaving off from the side walls and thus opening a passage for the water to the arch rings. Rectangular grooves are to be left in the side walls a few inches above the arch. The waterproofing is to be brought up the side walls and into these grooves. Then the grooves are to be filled flush with concrete.

Since both the arch and the side walls are founded on rock, they effectually seal the canal. There is, therefore, no object in placing backfill over the haunches or abutments and none is provided. However, there will be a covering of rock spoil, the same as at all places on this contract where the natural surface within the prism is below canal bottom.

Personnel

The plans for this work have been made in the office of Alex. E. Kastl, Special Deputy State Engineer, who, under the State Engineer, J. A. Bensel, has charge of plans and construction of the whole Barge Canal. The preparation of the plans on this contract has been under the immediate supervision of H. D. Alexander, Special Resident Engineer, while the aqueduct and steel work have been planned under the direction of W. H. Yates, Supervising Engineer.

Proposals on this contract were opened on Mar. 18, 1913, and the bidding was spirited, twelve firms submitting tenders. Nine of these were less than the engineer's estimate, which was $1,131,523, and the lowest was nearly 12% below this estimate. The work has been awarded to the low bidder, the Maryland Dredging & Contracting Co., of Baltimore, Md., at the contract price of $1,000,098.45.

❖

An Apprenticeship Course in Signal Engineering has been established by the Union Switch & Signal Co., at the Swissdale, Penn., works. According to Bulletin 66, issued by the company, the apprenticeship course covers two years and students in the course have an opportunity to become familiar with mechanical, electric and electro-pneumatic appliances, field installations, and the erection and design of various types of signaling apparatus. In certain cases, undergraduates in engineering schools may enter the apprenticeship course during their summer vacation and the time so spent will be credited to the course. Apprentices receive $65 a month the first year and $75 a month the second year. The company also pays the entrance fee and one year's annual dues including gymnasium charges for apprentices who wish to join the Westinghouse Club. Two weeks' vacation with full pay is given apprentices who have been in the employ of the company continuously one year or more.

A Belt Conveyor Was Used for Back-Filling a Sewer, in Grand Rapids, Mich. The work required that the earth be excavated for the laying of a 12-ft. concrete sewer, a portion of it replaced upon the finished sewer and the remainder loaded into wagons for hauling away. It was thought by the contractors, Carpenter & Anderson, that these three operations could be combined by the use of a specially designed conveying system. The excavation was performed by a Hayward orange-peel bucket, operated from a traveling derrick. The conveying system was erected beside the sewer and the work of back filling commenced. The outfit consisted of a 30-in. belt conveyor equipped with a traveling loading hopper and a traveling tripper. The total length was 210 ft., extending horizontally for the greater part of its length, and rising at its head to discharge into the wagon-loading hopper. All material from the bucket was delivered into the traveling hopper, which was moved to keep pace with the excavator. It was possible to advance the excavator about 50 ft. before it became necessary to move the conveyor. This method is claimed to have proved superior to others, both as regards the cost of the construction and the dispatch with which it was performed.

A Dipper Dredge with Hydraulic Jets for Leveling the Spoil Banks

By Chester B. Loomis*

A dipper dredge having some unusual features has been built for the Dominguez Estate Co., of Los Angeles, Calif., for cleaning the channel of the Los Angeles River where it flowed through the ranch and for building levees. The special feature is its equipment with hydraulic "giants" or nozzles for water jets to level the spoil banks or levees.

The Dominguez Rancho, one of the old Spanish grants in southern California, lies about 16 miles south of Los Angeles and contains about 20,000 acres. The eastern boundary of the ranch is the bed of the Los Angeles River, which at this point flows through a flat alluvial plain. The banks are very low and in recent years the portion of the ranch near the river has been subject to overflows due to the growth of tules and willows in the bed of the stream, which is practically dry in summer. Originally the stream bed was kept clear of growth by

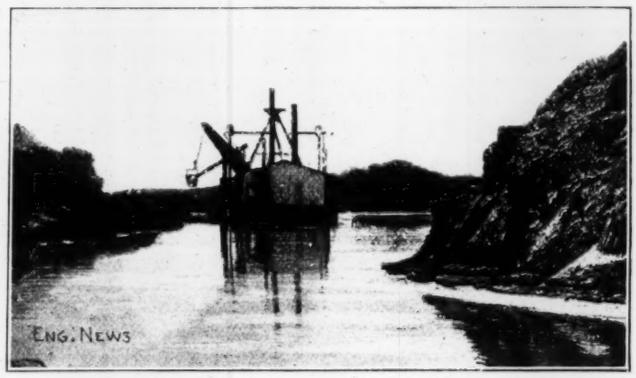


Fig. 1. Dipper Dredge Clearing the Channel of the Los Angeles River and Building Levees

(Projecting from the right side of the deck house is one of the hydraulic giants for leveling the spoil bank with a water jet to shape it as a levee.)

pasturing sheep and cattle, but as the land increased in agricultural value the stock was no longer kept along the river, and the resulting growth soon nearly filled the channel. This caused the winter flood water to overflow the land, decreasing its value.

The flood of the winter of 1910-11 was particularly heavy and the Dominguez Estate Co. decided to dredge the channel and build levees to confine the flood water to the channel. After careful study, a dipper dredge with vertical spuds was decided upon, and the contract was let to the Marion Steam Shovel Co. for the machinery for a dredge with a 1½-yd. bucket, a 55-ft. boom and a dipper handle of sufficient length to dredge 10 ft. below the water surface.

The hull is of steel, 75x32 ft., and 6 ft. deep, with three longitudinal and five transverse bulkheads, which extend to within 10 in. of the top of the sides. The tops of bulkheads are stiffened with 3x2-in. angles, and an angle of the same size runs completely around the hull on the inside. These angles formed a foundation for the 6x8-in. deck timbers, and the 2-in. deck planking was flush with the top of the sides. The bottom plates

*Consulting Mechanical and Hydraulic Engineer, 419 Wilcox Building, Los Angeles, Calif.

were stiffened with angles 5x3x½ in. and the sides with vertical angles 2x2x¼ in. A well was left in the center of the hull, running from 5 ft. back of the front plates to the midship bulkhead, to allow an alleyway for the cables and sheaves. Extra bracing was used under the turntable and around the spud casings, which were also built of steel and lined with 2-in. oak planks. Eye-plates

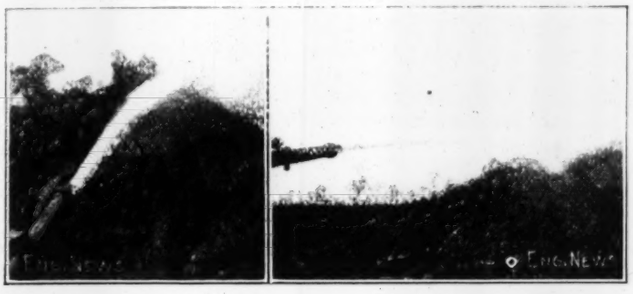


FIG. 2. HYDRAULIC GIANTS ON THE DREDGE, SLUICING THE SPOIL BANK INTO SHAPE AS A LEVEE

were riveted to the sides of the hull to take the A-frame backstay cables.

This hull proved very stiff and satisfactory and was lower in cost than a wooden hull. The plates were all punched and the angles riveted on where possible. The hull was erected and bolted together in the shop of a local iron works, and the parts then marked and shipped flat in sizes for wagon haul. After the plates were on the ground the entire hull was riveted up in ten days by 8 men and a foreman. Before the deck was placed, two steel tanks, each holding sufficient fuel oil for two days' run, were installed in two of the after compartments of the hull, one on each side. These were so piped that the oil could be drawn from either or both tanks as desired, and so arranged that both tanks could be filled from either side.

The machinery equipment is of the builder's standard type, except that there are special additions for the hydraulic "giants," as noted below. The hoisting engines are of the non-reversible type with cylinders 8x10 in., a 22-in. hoisting drum and a 15-in. backing drum; both drums are operated by outside friction bands set by steam rams. The swinging engine is of the same type, with cylinders 6x7 in., and is compound geared to a long shaft having at each end a 17½-in. drum for direct lead to the 15-ft. swinging circle. The spud hoist machinery consists of three 12-in. drums compound geared to cone frictions, the friction shaft being driven from the end of the backing shaft through jaw clutches. The spud hoist can be thrown out of gear when not in use. All the spuds are vertical, the two forward ones 20x20 in. and the stern spud 14x14 in. The boiler is of the locomotive type, with a 50-in. barrel and a firebox 54 in. long, the total length of boiler being 15 ft. The working pressure is 125 lb. With the exception of the hoisting and backing lines all of the ropes are led above the deck to the drums, in order to avoid interference with the hull framing. The deck house or superstructure has a wood frame and corrugated steel sheathing.

In addition to the regular machinery, a second locomotive boiler and a compound duplex outside center-packed Dow steam pump 9¼x10x12-in. were installed. The boilers were so piped that they could be run singly or together as desired. The suction of the compound pump was led to the stern, where the water was the clearest. The discharge was led forward along the deck (outside the deck house) to the spud casing, and from there was carried across the hull to the opposite spud casing. Just aft of each casing a platform about 4 ft. high was built and on it was mounted an hydraulic giant with a 2-in. nozzle connected to the pump discharge pipe. These giants were used to sluice down the spoil bank into the form of a broad flat levee.

The method of procedure was as follows: The brush was cut in advance of the dredge, and piled on each side of the line of the cut and back about 10 ft. As the channel dredged was 80 ft. wide it was found more economical to have the day shift make a cut half the width of the channel, piling the material on one side, over the brush. After making a cut and when the dredge was moved ahead for the next cut, the sluicing pump was started and the jet played on the spoil bank. As the bank was directly opposite the giant the water jet was very effective, and it was found that the bank could be cut down into a levee about 7 ft. high, with a flat top about 10 ft. wide in about half the time it took to excavate it with the dipper. This left the man who handled the giants free to attend to other duties.

At the end of the day shift the dredge was moved back so that the night shift could start the cut on the part not excavated, the same method of sluicing being employed. It was found that a water pressure of 30 to 35 lb. was the most effective. The men became very skillful in handling the giants, and took considerable pride in leaving a true flat top to the levee. The levee with the brush mattress under it proved very effective in withstanding floods with practically no erosion.

The spoil encountered was largely a sandy loam and silt with occasional seams of clay, and consequently washed very easily. It was found that when stumps were thrown up they could be washed back very quickly or buried by undermining with the jet and then washing more dirt over them. The dredge was designed by the writer, and was built and operated under his supervision as Superintendent of the Dominguez Water Co.

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Gathering of Atlanta, Ga., Engineers—Nearly a column of the Atlanta "Constitution," of July 13, is devoted to the proceedings of 75 of the city's engineer citizens—members of the American Society of Civil Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, American Institute of Architects, American Chemical Society, the Engineering Association of the South, all of which have local organizations in Atlanta, together with stray members of the American Institute of Mining Engineers, the National Electric Light Association, the American Society of Municipal Improvements, the American Water-Works Association and the American Public Health Association.

The site of the gathering was the city water-works pumping station, the time, Saturday afternoon, and the purpose was to enjoy "an old-fashioned Georgia barbecue, with all the trimmings." The chairman of the meeting was Allen M Schoen, President of the local branch of the American Institute of Electrical Engineers. Among the speakers were James Nisbet Hazlehurst, of the American Society of Civil Engineers, Park A. Dallis, of the American Society of Mechanical Engineers, and L. J. Hill, Jr., President of the Atlanta section of the Engineering Association of the South. The key notes of all the addresses was a desire for a closer affiliation of the local members of the various engineering societies and a wider participation of engineers in the activities of the community. The "Constitution" says in closing its account:

The meeting finally adjourned, subject to the call of the executive committee, and thus was launched an affiliated society of technical men, doubtless destined to add materially to the general advancement of the city.

Venturi Meter Coefficients

By Allen Hazen*

The Venturi meter, invented by Clemens Herschel, has taken its place as the most accurate and convenient means of measuring large quantities of water. As the years have gone by, its application has steadily widened. Continued experience with it has given increased confidence in the accuracy and reliability of the results. Nevertheless, errors in Venturi measurements are of sufficiently frequent occurrence to deserve attention. These errors may conveniently be divided into three classes:

(1) Those resulting from obstructions or defects in the Venturi meter tube. In a few cases large errors have resulted from irregularities of this kind, but they have been infrequent.

(2) Errors resulting from errors in adjustment of the indicating and recording apparatus. These errors have been too common, owing to the delicate nature of the apparatus that must be used and the lack of sufficient skill in the men available to take care of the apparatus. Progress has been made in making apparatus so substantial that it is less likely to get out of order, and arranging it so that the accuracy of its adjustment may be easily tested at any time, and still further improvements may be hoped for.

(3) Errors due to the use of erroneous coefficients or the incorrect application of the coefficient in the construction of the recording and indicating apparatus. It is to errors of this third class that the following relates.

A considerable proportion of the indicating and recording apparatus that has been built and that is now in service has been based upon a variable coefficient. That is to say, it has been assumed that the coefficient varied with the throat velocity of the water in the meter. In the writer's judgment there is no sufficient ground for using variable coefficients, but on the other hand, the best data available point strongly to the desirability of using a fixed coefficient for all rates of discharge. The errors involved in the use of a variable coefficient, while they have probably not exceeded 5% in the apparatus that has been furnished, are sufficiently great so that every proper effort should be made to avoid them.

As to Whether the Coefficient Is Constant

Mr. Herschel, in his original description of the Venturi meter, and of the experiments upon which it was based (Trans. Am. Soc. C. E., Vol. 17, p. 252) states:

> Though the areas of discharge were as 81 to 1 and the interior frictional surfaces were widely different, the resultant coefficients are at extreme points only 6.5% apart, and the deviation of any single experiment from the resultant mean is 3% in the case of 1-ft. tube, excluding the unreliable interpolated results, and only ½% in case of the 9-ft. trunk for its whole range of velocities. . . . Or by limiting the use of the meter to velocities greater than 9 ft. per sec. through the Venturi, being about 1 ft. per sec. through the pipe thereto appurtenant, all the ranges of variations above given become materially less.

In Herschell's "115 Experiments," in which these data are reviewed and other data are added, it is stated:

> Practically, we must use a coefficient of reduction, whose range is, however, surprisingly small.
>
> As will be seen from the diagram, this coefficient between the limits of 8 ft. velocity through the throat (0.9 ft. velocity in the pipe) and 28 ft. velocity through the throat (3.1 ft. velocity in the pipe) ranges only between 0.97 and 1.00 for all three meters.

Studying Mr. Herschel's experiments further, as can be most conveniently done by the use of Plate 3 of his "115 Experiments," it is seen that with velocities of more than 15 sec.-ft. there was a slight decrease in the coefficient in two series of experiments and a slight increase in the third. The differences were small and not in the same direction. In a similar manner there was a decrease in the coefficient with velocities under 12 sec.-ft. in one case, and with velocities less than 3 sec.-ft. in another case, and an increase with velocities less than 5 sec.-ft. in the third case. These differences only became marked at velocities that were relatively unimportant. They differed in direction and do not afford a sufficient basis for any presumption that there is a tendency for the coefficient to change with low velocities.

Mr. Freeman's exhaustive experiments on jets* and their coefficients are directly applicable because the jets that he used, as was pointed out by Mr. John C. Trautwine, Jr., in discussing his paper, are precisely equivalent to the operating end of a Venturi meter. The formula for calculating the discharge is the same, and the velocities that he used with nozzles were several times higher than are ever used with Venturi meters.

Freeman adopts for general use with properly constructed nozzles a coefficient of 0.995. This is for a nozzle with a slope of 10.5° to 15° convergence. (The convergence of a Venturi meter is 10.5°.) On p. 507 he states:

> If for a few inches back from the end there be given to the interior surface a smoothness equal to that which a good brass finisher gives to ordinary lathe-finished work, and if, moreover, the piezometer orifices are carefully made normal and flush with a surface parallel to the axis of the current, and withal so near the nozzle proper that no loss of pressure of importance can intervene; then, if the pressure is accurately measured, I see no reason why this coefficient may not be applied with a feeling of entire confidence that the result is within 1% of the truth.

Considering the question of the constancy of the coefficient of discharge with varying pressures, Freeman states:

> Starting with about 120 ft. head of water, and in the successive experiments gradually reducing this head to only a tenth part as great as that with which we started, the variation in the coefficient between the first condition and the last was less than 0.1%. For heads below 10 ft., some variation would very likely occur, but no reason appears why the coefficient for 200 or even 400 ft. head should not be the same as for 100 ft.

The conditions specified by Mr. Freeman for smoothness and accuracy of finish of the jets are above those ordinarily reached in Venturi meter tubes, and it would not be epected that as high coefficients would be found for Venturi meters as for nozzles. Otherwise there is no apparent reason why Freeman's observations on nozzles should not apply to Venturi meters. Certainly if there were a change in coefficient in Venturi meters with high velocities, these exhaustive experiments of Mr. Freeman would have indicated some corresponding change. Until there is definite evidence of a tendency of the coefficient to change with velocity, the writer believes that the most probable values for all velocities will be obtained by the use of the same coefficients found to apply at the velocities where data are adequate.

In studying these data the idea suggested itself to the

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*Of Hazen & Whipple, Consulting Civil Engineers, 30 E. 42nd St., New York City.

*"The Nozzle as an Accurate Water Meter," by John R. Freeman, Trans. Am. Soc. C. E., Vol. XXIV, p. 492.

writer that the head on a meter (or on a jet) consists of the difference between the velocity heads for the velocities at the entrance and at the throat, less the frictional resistance of the cone. The frictional resistance of the cone can be computed by the formulas used for the friction of water in pipes, by dividing the cone up into an indefinitely great number of very short parts and computing the velocity and friction in each. For ordinary water-works conditions the friction of the water in the cone computed in this way amounts to 2%, more or less, of the total head on the meter. Reducing the head by 2% reduces the discharge by 1% and the resulting coefficient would be 0.99.

With a 20-in. throat more exact calculations based upon this idea give the following results:

5-ft. velocity, C = 0.990
20-ft. velocity, C = 0.992
40-ft. velocity, C = 0.993

and with a 25-ft. velocity in all cases

with a 40-in. throat, C = 0.993
10-in. throat, C = 0.991
5-in. throat, C = 0.990
1-in. throat, C = 0.987

As to the Value of the Coefficient

Mr. Freeman's coefficient for nozzles of 0.995 is clearly too high for Venturi meters. Mr. Herschel's experiments indicated an average value between 0.98 and 0.99. Calculations based upon the assumption that the discharge is the theoretical discharge for the head less the frictional resistance of the cone indicate slight changes with velocity and with diameter, and an average value of about 0.990 for ordinary water-works conditions. The three lines of data are very consistent and indicate that the round figure 0.99 may be used for ordinary purposes and for all velocities.

In addition to the data noted there have been tests of Venturi meters and their coefficients from time to time. It is believed that the most important of these data have not been made public. The writer has no reason to believe that any of them indicate values substantially different from that deduced above, or that they warrant the use of variable coefficients.

Manometers

The surest way to test the accuracy of a Venturi meter is to use manometer tubes, which may be conected with the same pipes that serve to connect the Venturi tube with the ordinary indicating and registering apparatus. In filter plants, and for other corresponding services where the water is not under much pressure, the direct difference in water level may be observed. The water level may be raised equally in glass tubes connected with the two points of the meter by connecting them together at the top and sucking air out until the water is brought to a convenient level for observation.

Chester M. Everett of our office has designed an apparatus which is convenient where such observations are to be frequently made. A water ejector draws air from the top of a manometer, and is arranged so that air is drawn until the water in a control vessel, connected with the same pipe that connects with one arm of the manometer, reaches the level of zero in a scale for that arm, at which point air ceases to be drawn. By turning on the water when the apparatus is to be used, this automatically produces the desired degree of suction to bring the water on one side of the manometer to the zero mark and hold it there. The head on the meter can be read directly from the water in the other column on a graduated scale. Where water is under considerable pressure, and where the velocities are greater, mercury manometers are used.

Manometers are frequently made with scales graduated to show the rate of flow through a particular meter tube. If an erroneous coefficient has been used for graduating both the manometer and the registering apparatus, the error will not be disclosed by comparison. The writer prefers to have all manometers to be used for test purposes graduated in inches or feet. The rate of flow is computed by the formula that follows. A single observation of the manometer will show whether the indicator of the regular instrument is showing correctly at that time, and a series of observations at stated intervals will serve as a basis for checking the returns of the register.

Formulas

The formula for the flow in a Venturi meter, in its most convenient form, is

$$Q = 28{,}276\, C \frac{d^2}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{h}$$

where Q is the quantity in United States gallons per 24 hours, d and D are diameters of the throat and entrance respectively in inches, and h is the head on the meter in feet of water.

For $C = 99$, which is the value which I prefer to use, this becomes for h in feet of water

$$Q = 27{,}993 \frac{d^2}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{h}$$

and for h in inches of water,

$$Q = 8081 \frac{d^2}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{h}$$

and for h in inches of mercury

$$Q = 28{,}684 \frac{d^2}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{h}$$

The specific gravity of mercury under water is taken as 13.6 — 1.0 = 12.6.

In applying this formula to a given Venturi meter tube, all the constants may be multiplied together once for all. The formula will then be reduced to the form

$$Q = n \sqrt{h}$$

where n is the product of the above noted factors. For instance, with a 42-in. meter with a 16-in. throat and a mercury manometer, we have

$$Q = 28{,}684 \frac{16^2}{\sqrt{1 - \left(\frac{16}{42}\right)^4}} \sqrt{h} = 7{,}420{,}000 \sqrt{h}$$

With 1 in. of mercury the flow will be 7.42 million gallons daily, and for other rates in proportion to the square root of the manometer reading. This formula can be solved with sufficient accuracy on an ordinary 10-in. slide rule.

Testing out Venturi meters by a manometer in this way at not too infrequent intervals will add to the accuracy and reliability of the results.

[Clemens Herschel and J. W. Ledoux have responded to invitations to discuss Mr. Hazen's paper in this issue. Their letters follow immediately.—Ed.]

Discussion of Mr. Hazen's Article by Clemens Herschel and J. W. Ledoux

Sir—Responding to the generous opportunity given me to discuss in this issue Mr. Allen Hazen's paper with the above title, let me say, in the first place, that the engineering profession has been only too slow to take up the invitation extended to it by me in 1887, to make "further accurate experiments" with the Venturi meter, "in the general planning or arranging of which, I hope to be allowed to make some suggestions."

So far as I know, the only three *series* of experiments to test the Venturi meter for accuracy ever made, are to be found described on Plate 3 of "115 Experiments," referred to by Mr. Hazen; and two of these series were based on weir measurements; not a precise check on Venturi meter measurements. The only precise such check or test that can be applied, is or would be one furnished by a *tank* measurement.

This is not intended as a criticism upon the action of engineers who have considered this subject during the past 26 years. They have not had the opportunity to make tank measurements for the purpose indicated; and as has often been pointed out, from the time of Galileo down to the present time, while observatories for the pursuit of astronomical recreations abound, and wealthy men vie with each other in endowing them, and still others, the eminently useful and beneficent art of measuring water is lacking of endowed hydraulic laboratories in which such questions as Mr. Hazen's article sets forth should have been long ago definitely settled, or may yet properly be investigated and spread upon the public records.

Still less is it possible, on the basis of analogies, to compute with precision a hydraulic coefficient—so that after 26 years presence among us of the Venturi water meter, and at a time when over 5000 of them are in use (literally, the wide world over, ranging in size from ¼ in. diameter of main pipe to 18 ft. diameter), we are still relegated for tests such as Mr. Hazen's query can call forth to the tests of meters published 26 and 16 years ago.

How the results of nozzle gagings can affect the answer now sought, the writer fails to see. A nozzle is a form of orifice. The formula for calculating its discharge is in the form usual in the case of orifices; while that used for calculating the discharge of a Venturi water meter, is based on the *difference* between two pressures, the one observed at the throat, the other at the upstream end of the meter. Going further, one may also point out that without a registering or recording apparatus attached, the form of tube the present writer invented in 1886, to be used in "a new method of gaging water," is not a "meter"; and still less is an orifice that wastes water upon the ground, or elsewhere, and whose quantity must be computed, a "meter."

Another point: 26 years ago, the Venturi water meter was a newcomer. It had to fight for the right to live. Doubts of its possibility or expediency for practical use, or of its accuracy when used, abounded in the technical press, and even in the transactions of learned societies. Ten years went by before anyone besides the inventor used the meter; and then it was not an engineer, but the lawyer-president of a little water-supply company in Plainfield, N. J., who ordered the first meter put in, outside of works under charge of the writer. "On trial," of course, but that was much better than ignoring the meter entirely, or decrying it.

Under the circumstances, and when dealing with the sale of water, the use of the meter was thus distinctly one of commercial policy; and as such it was expedient, or even necessary *never to have the meter read too much*. It was impracticable to make or await further experiments for the sake of establishing a still nearer approach to the truth, and the inventor pleads guilty to having always encouraged the use of coefficients, especially when making out bills for water drawn, which should assuredly not charge too much.

After ten years of such work at Holyoke, Mass., and eleven years more of it in New Jersey, constantly beset with obstreperous customers of no mean order of financial and intellectual ability, and yet never having had a water bill successfully disputed; on the other hand, having at the same time and under such exercise of the art of measuring water built up properties worth many millions of dollars to his employers; the writer cannot say that he should have done differently.

But now the situation is different. Some 5000 Venturi water meters are giving satisfactory service from West Australia, around the globe, back to West Australia; and in many cases it is not a question of making out bills that cannot successfully be disputed, but, instead, to keep records that shall tell the truth. And in this situation, no one will more cordially welcome Mr. Hazen's article, and the possible incentive it will give to the making and publishing of a comprehensive group of series, each of many experiments accurately made with different-sized meters, than the writer. Single tests, at single velocities, have frequently been published. But meters of many sizes, each one tested by tank measurement on a series of velocities, are yet to be experimented with, and the results of the experiments published.

Clemens Herschel.

2 Wall St., New York City.
May 31, 1913.

Sir—Mr. Hazen's paper on Venturi tubes is very interesting, but his essential statements do not correspond with the experience of the writer.

Mr. Hazen takes the ground that there is not sufficient evidence of a variable coefficient in Venturi tubes. For such high velocities as he has under consideration his statement is approximately true. For a tube having a ratio of 2 to 1 in diameters the coefficient is practically constant from 2 ft. per sec. upwards, although there is a slight rise with the increasing velocity. For example, in a particular tube, the coefficient at 2 ft. per sec. velocity was 0.97; at 10 ft. velocity it was 0.978. In a second tube with the same ratio, the coefficient at 2 ft. velocity was 0.967, and at 8 ft. velocity 0.977. In a third tube the coefficient at 2 ft. velocity was 0.98, and at 8 ft. per sec. 0.99. In a fourth tube the coefficient at 2 ft. velocity was 0.97. However, taking these same tubes, the coefficient C for the first tube at 1 ft. velocity was 0.965 and at ¼ ft. velocity, 0.93. For the second tube at 1 ft. C was 0.956 and at ¼ ft. velocity, 0.923. Third tube at 1 ft. velocity C was 0.977 and at ¼ ft.

velocity 0.95. Fourth tube at 1 ft. velocity C was 0.96 and at $\frac{1}{4}$ ft. 0.955. These statements are based on so many individual determinations as to leave no doubt that the coefficient varies slightly at high velocities and materially at low velocities. In fact, we made a very large number of determinations at velocities below 2 ft. per sec. and especially below 1 ft. per sec. through the main. In making these experiments various types of manometers were used; usually two different types were used simultaneously with each operation. To multiply the readings colored tetrachloride of carbon was used and also oil, and the specific gravity of the former was varied so as to exaggerate the readings when necessary; but it was found that the ordinary water manometer with an air column gave about as satisfactory results as any of the others, because in that case the meniscus is very much more clearly defined, and the difficulties due to the oil or tetrachloride clinging to the glass tubes were eliminated. Some of the best and most reliable results, however, were obtained by using two open tanks, one connected with the throat and the other with the full section of the main, and hook gages for determining the level of the water. There was always great difficulty with any of these devices in absolutely eliminating the possibility of air, a minute quantity of which will vitiate the results.

No. 1 tube was nominally 4 in. by 2 in., actually 3.928 by 2 in., and clean. No. 2 tube, the same as last, was nominally 4 in. by 2 in., but had been in use for several years and was dirty. No. 3 tube was 7.949 in. full section, 3.813 in. throat and clean. No. 4 tube was nominally 16 in. by 8 in. The average of all these tubes for high velocities has been taken as 0.977, and when this is multiplied by the square root of $2g$ the convenient resulting formulas in use by the writer for several years are:

$$V = \frac{7.836\ C\ H}{\sqrt{\left(\frac{D}{d}\right)^4 - 1}} \qquad (1)$$

in which

V = velocity in feet per second through the main;
D = diameter of the pipe;
d = diameter of the throat in any unit, usually taken in inches;
H = venturi head in feet;
h_g = head in inches of mercury.

To obtain gallons $Q = 3530\ V\ D^2$, in which

Q = U. S. gallons per 24 hours;
V = velocity in feet per second;
D = diameter of the main in inches.

Substituting for V its value in this formula we get

$$Q = \frac{27,000\ D^2\ C\ H}{\sqrt{\left(\frac{D}{d}\right)^4 - 1}} \qquad (2)$$

in which

Q as before = U. S. gal. per 24 hours.

If we take the Venturi head in inches of mercury, h_g, these formulas would reduce to

$$Q = \frac{25,200\ D^2\ C\ h_g}{\sqrt{\left(\frac{D}{d}\right)^4 - 1}} \qquad (3)$$

and when it is desired to determine the ratio of the tube when the quantity of the water or velocity is known:

$$\frac{D}{d} = \left(1 + \frac{61.4\ H}{V^2}\right)^{\frac{1}{4}} \qquad (4)$$

$$\frac{D}{d} = \left(1 + \frac{765,000,000\ D^4\ H}{Q^2}\right)^{\frac{1}{4}}$$

Mr. Hazen's suggested arrangement of formula is not convenient for handling the ratio of the Venturi tube, and that is because he assumes a velocity through the throat instead of through the main.

It is probable that the high coefficients determined by Freeman's test on nozzles are due to smoothness of bore. A Venturi tube, as ordinarily made, has a bronze-lined throat but the balance is a casting. The writer has made determinations both with Venturi tubes furnished by the Builders Iron Foundry and by the Simplex Valve & Meter Co., and believes that the coefficients are generally below 0.98. He has also made experiments to determine whether the coefficient was affected by pressure and these results showed no material difference between the flow through a Venturi tube at atmospheric pressure and at 40 lb. or more per sq.in.

The results above given, showing the coefficients of Venturi tubes, are general, but the writer expects to publish details of these experiments giving all the methods and results upon which these general statements are made.

The following table is an illustration:

Gage reading, ft.	Head in ft. of water	Lb. per run	Time min.	Time sec.	Velocity ft. per sec.	Coefficient of discharge	Remarks
							May 14, 1912.
0.073	0.0073	600	11	20	0.168	0.915	Carbon tetrachloride. Differential gage. Specific gravity 1.10.
0.073	0.0073	300	5	10	0.168	0.915	
0.155	0.0155	600	7	41.1	0.248	0.927	
0.085	0.0255	600	5	56	0.21	0.926	
0.095	0.0285	300	2	47.2	0.242	0.942	
0.095	0.0285	300	2	36.8	0.242	0.942	
0.16	0.048	600	4	15	0.348	0.952	May 14, 1912.
0.16	0.048	300	2	07.2	0.348	0.952	Carbon tetrachloride. Differential gage. Specific gravity, 1.30.
0.365	0.1094	700	3	18.4	0.672	0.946	
0.365	0.1094	700	3	18	0.673	0.945	
0.880	0.264	700	2	04.2	1.072	0.972	
0.875	0.262	700	2	04.6	1.070	0.973	
2.43	0.729	700	1	14.6	1.790	0.976	
2.45	0.735	700	1	14.8	1.783	0.968	
0.305	0.305	5000	13	52.6	1.142	0.965	May 18, 1912.
1.310	1.310	8000	10	29.8	2.53	0.975	Air differential gage.
2.990	2.990	11000	9	41.4	3.61	0.970	
0.422	5.31	14000	9	13.2	4.83	0.975	May 18, 1912.
0.665	8.24	28000	14	49.2	6.005	0.973	Mercurial differential gage. Specific gravity, 13.58
0.967	12.17	28000	12	09.6	7.32	0.976	
1.324	16.66	28000	10	25	8.54	0.975	
1.740	21.87	28000	9	1.8	9.83	0.977	
0.320	0.329	5500	14	55	1.172	0.935	May 25, 1912.
1.296	1.296	12000	16	07.8	2.365	0.997	Air differential gage.
2.982	2.982	15000	13	18.4	3.585	0.966	
4.499	4.499	20000	21	32.5	4.42	0.969	
0.651	8.37	20000	15	50.7	6.02	0.969	May 25, 1912.
0.966	12.14	30000	13	06.7	7.27	0.973	Mercurial differential gage.

Diameter of full section mean of 6 readings 3.928 in.
Diameter of throat section mean of 3 readings 2.000 in.

(All results computed on a 10-in. slide rule).

$$V = 0.1905\frac{P}{T}$$

$$C = 4.05\frac{V}{\sqrt{H}}$$

in which

T = sec.
P = pounds
V = velocity
H = head in feet
C = deduced coefficient.

Heretofore the Venturi tube has generally been designed with a large ratio between the diameters of the full and throat sections, usually 3 to 1, and under these circumstances, when flowing at the rate of 6 ft. per sec. through the main, which is frequently desirable, the throat velocity would be 54 ft. per sec., which would cause a friction of about 6 ft. If the ratio of diameters were 2 to 1, the friction would be only 1.3 ft. Therefore, it is seen how desirable it is to keep the ratio of a Venturi tube low, and the meter register which is best adaptable to low ratios is desirable.

The formula for friction through the entire Venturi tube is

$$F = 0.0021\ V^2$$

where

F = the loss in head through the entire Venturi tube and

V_1 = the velocity in feet per second through the throat.

J. W. LEDOUX.

112 North Broad St., Philadelphia, May 27, 1913.

[The communications from Messrs. Herschel and Ledoux were submitted to Mr. Hazen. His reply follows.—ED.]

The writer agrees with Mr. Herschel that it is unfortunate that more results of definite tests of Venturi meters with tank measurements are not available. The importance of an accurate knowledge of the coefficients certainly justifies more experimental work than has been made public.

Mr. Ledoux has made welcome addition to data of this kind, although relating to a meter so small in size that their application to large meters in ordinary use must be made with caution. It may be hoped that the data yet to come will relate to larger sizes.

Mr. Ledoux agrees that for such high velocities as the writer had under consideration (that is to say, such velocities as are used in practice), the statement that there is not sufficient evidence of a variable coefficient is approximately true. His experiments indicate only a slight increase in the coefficient with velocity, the total change with throat velocities above 3 ft. per second being within the apparent limits of error of the experiments.

It is interesting to note in connection therewith that another maker of Venturi meters has used coefficients decreasing with increasing velocities. In other words, a sliding scale has been used, but sliding in the opposite direction from that in which Mr. Ledoux believes the coefficients to slide. The writer knows of no experimental data sufficient to support either practice; and the fact that different makers of Venturi meters hold diametrically opposite views upon this point does not lend support to either extreme view.

The decrease in coefficient at very low velocities mentioned by Mr. Ledoux, if it exists, is a matter of scientific interest rather than of practical importance, so far as the Venturi meters that have been used are concerned, for they have been based upon the use of higher velocities. These experiments at lower velocities are particularly interesting, and it is to be hoped that they may be repeated with full-size meters, in which, if the phenomenon proves to be analogous to that of the critical velocity in small pipes, the velocity at which the change in coefficient begins will be found to be quite different from what it is in the miniature meter.

The coefficient for small meters seems clearly to be less than for large ones. The average value found by Mr. Ledoux for a 2-in. throat is about 0.973, and would correspond with a somewhat higher coefficient for a full-size meter. The writer thought a few years ago that the most probable value for large meters was in the neighborhood of 0.98. Experience in the last two years has led him to believe that the use of 0.99 is more likely to yield accurate results for the sizes and velocities common in water-works practice, and that it is best to use this value until more exact experimental determination on full-sized meters is available.

Mr. Ledoux's formula proves upon trial to be identical with that suggested by the writer. One can be deduced from the other, and a coefficient applicable in one can be used without change in the other. The choice of form is, therefore, only a matter of personal preference.

ALLEN HAZEN.

Experiments on Uplift Pressure in Masonry Dams

By C. R. WEIDNER*

Since the failure of the Austin dam in September, 1911, there has been considerable discussion among engineers as to what provision should be made for uplift pressure in designing masonry dams. The views of some of the foremost engineers of this country were brought forth in a discussion of a paper† on this subject, presented at a meeting of the American Society of Civil Engineers. All recognized the existence of uplift and that it should be considered in the design, but a diversity of opinion existed as to the amount to be provided for. This, however, is a matter which is largely dependent on the experience and judgment of the engineer, as the uplift pressure may vary within wide limits and its determination in advance is so uncertain, that a large factor of safety must often be allowed. The engineer should, therefore, have available experimental data upon which to base his judgment but heretofore such data, if any existed, have not been published.

The writer was, therefore, interested in reading an article by R. Schaefer in *Zeitschrift für Bauwesen*, Vol. 63, 1913, p. 102, which describes and presents the results of some experiments to determine the uplift pressure at the Oester and Neye dams in Germany. The results of these experiments, when properly interpreted, should be of value to the designer of future structures. With the object of making these data available to those who may not have access to the original article, an abstract has been prepared giving the substance of the article relating to the experiments.

The Oester dam, on which the most accurate and extended investigations were made, is a power and water-supply dam located in the valley of the Ruhr near Duisberg, Germany. It is a masonry structure holding back 103 ft. of water and has a maximum height of 131 ft. It is 14.8 ft. wide on top, 87.0 ft. wide at the foundation, and of gravity section as shown in Figs. 1-5. The Neye dam, in the same part of Germany, is near the city of Remscheid for which it forms a water-supply reservoir. It, too, is a gravity masonry structure of about the same dimensions as the Oester dam. Its lower section is shown in Fig. 7.

OESTER DAM

In building the Oester dam provision was made for measuring the uplift in the foundation joint, by placing pipes along four sections (Fig. 1) leading from the foundation to the two galleries, which extend transversely through the dam. These pipes were 2.36 in. in diameter and were placed in a vertical position on the foundation (Fig. 6). They rested on a bell-mouth pedestal with a 4-in. opening, and at the elevation of the galleries the pipes turned with a right angle and extended horizontally into the gallery. A cock and Bourdon gage were attached to this end, by means of which the pressure in the foundation joint was determined, the vertical distance between the two ends being known.

The foundation of this dam is described as being com-

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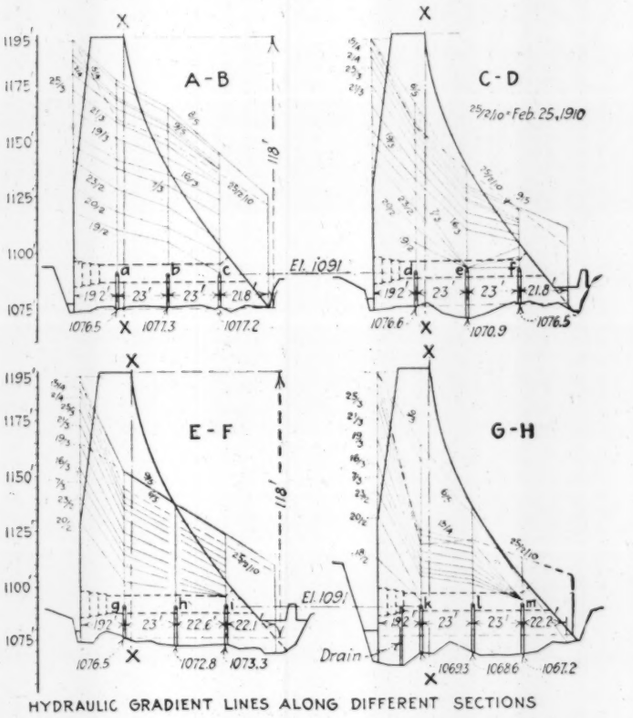
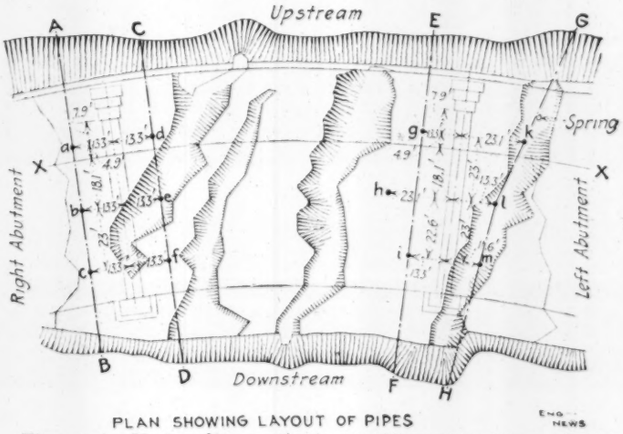
†Trans. Am. Soc. C. E., Vol. LXXV, 1912, p. 142. Paper by the late C. L. Harrison.

posed of graywacke and shale, which varies in character at different parts of the site. Firm benches of graywacke (at *g*, Fig. 1) vary with rotten shale (at the left abutment and at *a*, *b*, *c*, *d*, Fig. 1) and with strong but fissured shale banks (at the right abutment), the fissures being filled with clay. The stratification planes run about in the direction of the crevices shown in Fig. 1,





Figs. 1-4. Hydraulic gradients in different sections of dam at various dates.
Fig. 5. Plan of dam base.

FIGS. 1 TO 5. THE OESTER DAM

or approximately parallel to the valley in which the dam is situated. In the bed of the valley the strata stand almost vertically next to the crevices, with but little change in the angle of incidence; near the abutments they fall into the crevices. In some parts no regular direction of the stratification planes could be observed, as the entire site was pretty well broken up.

The deepest crevice at the foot of the left abutment was filled with blue clay, parts of which had turned to shale. In excavating this a spring was discovered near the upstream end (Fig. 1), the flow of which was intercepted and conducted to the left gallery. It discharged 19 gal. per hr. before the reservoir was filled and 870 gal. per hr. with the highest head on the dam. The crevice was excavated to a depth of 13 ft. below the mean foundation level at its upstream end, and to about 8 ft. over the rest of the area. After placing pipes *k*, *l*, *m* in the positions shown in Fig. 1, the space was filled with concrete. Pipes *a*, *b*, *c*, *d*, *f*, *g*, *h*, *i* were placed on perfectly dry parts of the foundation bed, while pipes *e*, *k*, *l*, *m* were placed in the crevices. The latter, however, were not placed directly in the path of the largest flow but at one side.

The gates of the dam were closed on Feb. 16, 1907, and on May 6, the water discharged over the spillway for the first time. A record of the pressures registered for different stages of filling was kept, and the resulting hydraulic gradients have been drawn in Figs. 1, 2, 3 and 4. Each gradient has been extended to the upstream face to indicate the corresponding water level in the reservoir. The probable extension from the downstream piezometer to the toe has only been drawn for the highest gradient, corresponding to the normal head of 118 ft.

A comparison of the gradients at the different sections

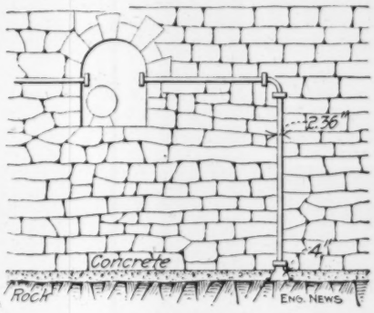


FIG. 6. METHOD OF PLACING PRESSURE GAGE PIPES IN OESTER DAM

shows a wide variation. In sections *A-B*, *C-D*, *G-H* the maximum gradients indicate that the uplift at the heel is equal to the full static head, while in section *E-F* the pressure at this point is indicated by the dashed extension of the gradient. The highest pressures were observed in section *A-B*, where the gradient indicates that the pressure varies uniformly from full head at the heel to approximately one-half the head at the toe. The smallest pressures were observed in section *G-H*, although this section runs along the crevice in which there was considerable water. The pressure may, however, have been relieved in this section by the drain from the spring. As section *A-B* is placed on the shallowest foundation and *G-H* on the deepest, the conclusion is drawn that the deeper foundations are conducive to a decrease in the uplift pressure, for the reason that more solid and impervious rock is reached. The pressures in sections *C-D* and *E-F*, whose foundations are also deeper than that of section *A-B*, verify this conclusion.

As the water was withdrawn from the reservoir during the growing period, the pressures in the piezometers were again noted and it was found that they decreased proportionally with the head in the reservoir. During the replenishing period the pressures rose again, in most of the pipes, however, the highest pressures recorded in the previous season were not again reached. In the succeeding years a decrease was shown from year to year. The highest pressures obtained in 1910 are shown by the dashed gradient lines in the figures. Only in piezometers *e* and *m* are the pressures in 1910 greater than in 1907.

The behavior of section *G-H* is particularly striking. From May 1 to 6, 1907, the water in the reservoir rose 3.64 ft., whereas, during the same period, the pressure in piezometer *k* increased 32.8 ft., in piezometer *l* 13.8 ft. and in piezometer *m* 9.8 ft. It seems that here a case actually occurred in which a seam was rent by the uplift, enabling the water to penetrate more freely and thus increase the pressure. On May 6, 1907, the pressures in sections *A-B* and *C-D* reached their highest values and began to decrease thereafter, while in sections *E-F* and *G-H* the maximum values were not reached until May 9. In 1908 the maximum pressures in piezometers *k* and *l* corresponded approximately with those of May 6, 1907, while the pressure in piezometer *m* increased 11.8 ft. above the highest recorded in 1907. In 1910, the pressure in piezometer *k* decreased 19.8 ft., in piezometer *l* 16.7 ft. and in piezometer *m* 3.9 ft. below the maximum value observed at any time previous thereto. The decrease in pressure, in 1908 and 1910, may be due to the fact that during these years the spring near piezometer *k* discharged 79 gal. per hr. more than in 1907.

Upon removing the gages from the pipes and opening the cocks, the water in nearly all of them squirted out with a good sized stream and with considerable force. In piezometers *a, c, e, f, h, k, m*, the pressure dropped to zero in from 1 to 5 min., and the pipes discharged but little if any water thereafter. Piezometers *b, d, g, i, l* showed a decrease of only 11.5 to 52%, and the water continued to flow in considerable quantities even after they had been open 3 hours. In the latter group the original pressures were attained in from 9 to 21 min. after the cocks were closed, while in the former group some of the piezometers attained the original pressure in a few minutes, some after several hours and in others the original pressure was never reached again. These observations were made with a head of from 65 to 70 ft. in the reservoir.

In 1907, after the reservoir was filled for the first time, it was noticed that considerable seepage occurred in the penstock trenches at the foot of each abutment. This amounted to 2613 gal. per hr. at the right abutment and 1162 gal. per hr. at the left. The flow soon decreased rapidly, although the water level in the reservoir fell but slowly. In 1908 the flow at the right abutment had decreased to 1742 and in 1910 to 1308 gal. per hr. Attempts to measure the flow at the left abutment, after the first year, were not successful. It was thought possible that seepage still continues to flow in the penstock trenches, the measurements, however, indicate clearly that in the course of time the seepage became smaller, due to the deposit of silt on the upstream face of the dam and the filling up of the pores in the masonry. The decrease in the uplift pressures no doubt may also be attributed to this cause.

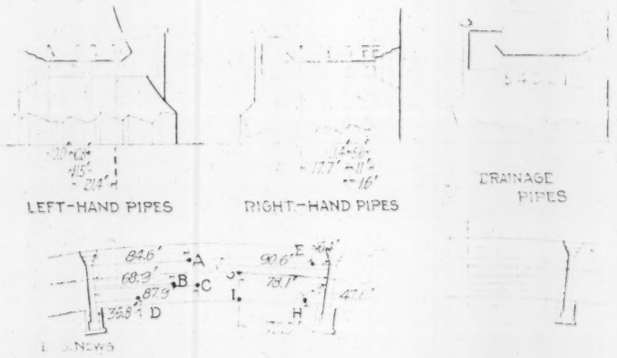
Neye Dam

Similar experiments were made at the Neye dam. Unfortunately the piezometer pipes (Fig. 7) were not set according to the designer's instructions when the dam was built, so that the hydraulic gradients for the different sections could not be drawn. The foundation for this dam consists of firm benches of graywacke and shale, which run almost at right angles to the valley and incline at an angle of from 15 to 35° toward the downstream side. Several small fissures, a few inches in width, were discovered near the left abutment and at the middle of the foundation. After excavating as far as possible the earthy oxide of manganese with which they were filled, these fissures were grouted. Pipes *a, b, g, e* rest upon these fissures, the others on sound rock.

In February, 1910, after the reservoir had been filled for some time, the following uplift pressures were observed with the water in the reservoir 102 ft. above the mean foundation level.

Piezometer	a	b	c	d	e	f	g	h	i
Pressure in ft. of water	57.8	54.1	49.3	31.6	46.0	56.9	54.0	44.7	32.8

These observations also show a decrease in the uplift proceeding from the heel to the toe. The pressure is, however, relatively less than at the Oester dam, especially at the heel. The maximum pressure at the Neye dam, at a distance of 14 ft. from the heel, is about 57% of the total head, while at the Oester dam, at a distance of 19 ft. from the heel, it was 85.5% in 1907 and 71.2% in 1910. As the foundation bed of the Neye dam is on an average of 26.2 ft. below the natural surface, while that of the Oester dam is only from 13.1 to 14.8 ft., the difference in the observed uplift pressures would corrob-



Figs. 7 to 10. Details of Neye Dam.

orate the conclusion stated above that a deep foundation decreases the uplift. At the Neye dam the small angle of inclination of the strata toward the downstream side is no doubt more conducive to the formation of uplift than their direction.

A system of drainage pipes (Fig. 7) was also installed at this dam for experimental purposes. It was not a foundation drainage but was placed within the dam, at a distance of 26.2 ft. above the foundation. It extended only between the center of the dam and the left gallery, into which five 3-in. pipes discharged. The maximum discharges obtained from the pipes in 1909 were:

Pipe 1	6.6 gal. per hr.
Pipe 3	1.6 gal. per hr.

The seepage from pipe 2 was only in drops and from pipes 4 and 5 there was no discharge at all. No rise in pressure was observed in any of the neighboring pipes, after a pipe was closed for a period of 84 days.

With reference to this drainage system, the opinion was expressed that an appreciable decrease in uplift would not be obtained, even if it were placed on the foundation and not at the elevation of the gallery. The reason given is, that as the system can only discharge at the elevation of the gallery, the pressure due to the difference in elevation between the gallery floor and foundation bed, would always exist. Hence, a drainage system would only be of use near the abutments, where the

foundation bed is at a higher elevation than the gallery floor and free discharge is provided for the seepage. This opinion is evidently based on the conditions existing at the Neye dam and necessarily would not hold in general. Even in this case an effective drainage system at the foundation, discharging into the galleries, would have considerably decreased the uplift pressures.

Conclusions

The following conclusions were drawn from the experiments:

(1) That water flows in numerous veins beneath the dam and penetrates into the finest cracks, seams and cavities, causing uplift pressure of various degrees of intensity.

(2) That the full static pressure does not act over the entire base but the resistance, which the water encounters in its flow toward the downstream side, gradually uses up the head and makes the pressure at the toe less than at the heel.

(3) Deep foundations are conducive to decreasing the uplift pressure.

These conclusions simply substantiate the views already held by engineers, who have had experience in the construction of dams. The question how large should the pressure be taken and what is the effective area upon which it acts, must be left to the judgment of the designer, as it depends principally on the character of the foundation.

In regard to the magnitude of the pressure, the writer agrees with the author that the largest pressures observed in the experiments should be used as a basis for estimating the uplift pressure in designing future structures, where the character of the foundation is similar to that at the two dams described above. It seems safe to assume, that only in exceptional cases would the intensity of uplift be equal to the full static head at all parts of the base. Furthermore, computations on the stability of existing structures, which would fail if this condition were fulfilled, bear out this assumption. It is, therefore, recommended that the full hydrostatic pressure due to the maximum head be assumed at the heel and one-half that pressure at the toe, the pressure being effective over the entire area and decreasing uniformly from the heel to the toe.

The opinion is also expressed, that the pressure may possibly act on an effective area equal to the entire area of the base, the action being explained in the following manner. In most dams a layer of concrete is first deposited on the foundation, and the fact is well known that concrete is not impervious. Then, if the percentage of voids between bedrock and concrete be assumed as only 40% of the area, it is conceivable that the voids, existing in a layer of concrete a short distance above, may be in communication with the lower ones and arranged in such a way, that the areas upon which the pressure becomes effective may lie directly above those in the lower layer not affected by the pressure. With the water penetrating into several such layers at small distances apart, it is possible that the horizontal projections of the different areas upon which the pressure is effective, would be equal to the area of the base. The pressure would thus become effective over the entire area. The writer believes that this condition, although possible, is highly improbable with good materials and workmanship in mixing and placing the concrete. The recommendation is, however, on the safe side and should be used for conservative designs. That the pressure was widely distributed over the area is shown by the fact, that some pressure existed at every one of the 21 points at which the piezometers were placed.

Stress is laid on the fact that it is expedient to place the dam not only on a firm foundation but also as deep as possible in the rock, not only as a protection against sliding but also to lessen the uplift pressure. No reference is made to a cutoff wall or under drainage, as a means of minimizing the uplift, which, in the opinion of the writer, would be a far more economical way. The suggestion is made that all interstices should be carefully grouted, and the masonry laid in a bed of rich mortar directly on the foundation bed, without a layer of concrete in between.

While these experiments indicate what might be expected in regard to the intensity of uplift pressure under similar conditions, it must be borne in mind that every structure of this kind is a problem unto itself, and that no fixed rules can be laid down which will be applicable to all cases. The late C. L. Harrison summarized the situation admirably when he said:

> After all, determining the proper allowance to be made for the uplift pressure is not an exact science, but the engineer, after considering all the obtainable facts bearing on the case, must use his best judgment, based on experience and observation, in making these allowances; and the final design should be made so as not to involve undue risks on the one hand, or unnecessary cost on the other.

Similar observations on other dams would be very desirable to supplement the information obtained. The cost of installing the apparatus would not be large; it amounted to about $150 at the Oester dam. Each section should have at least five piezometers, the two outer ones being placed about 3 ft. inward from the heel and toe.

❖

A Speed of 3310 r.p.m. for an automobile motor was established by the automobile testing plant of the Worcester Polytechnic Institute, Mass. The conditions under which the tests were made approximated, as closely as possible, actual duty on the road. A dynamometer indicated that the 20-hp. rating of the motor, a Studebaker, was reached at 1120 r.p.m. At 1900 r.p.m., 30 hp. was developed, and at 3310, the maximum, 36 hp., equivalent to a speed of 89 mi. per hr. At about 3300 r.p.m. the power curve began to drop.

❖

The Collapse of Piers and similar structures, which are often built without proper designing and without regard to the possible heavy loading to which they may be subjected by crowds, etc., is a class of accident which is to be guarded against in Germany by means of government inspection and supervision. Last year a steamship landing pier at Binz, on the island of Rugen, gave way under a crowd, and a number of persons were drowned. According to a statement in "The Engineer," of London, the government authorities called for reports from numerous local bodies and have issued a circular containing regulations introducing a stricter control over such structures.

It is provided that the more important landing piers must be thoroughly examined at least once a year to see if they are secure. The permission of the Government must be obtained for the construction of such piers, and the party building the pier must demonstrate its reliability with regard to appropriate construction and maintenance, or must furnish proof of the safety of the structure at certain periods, and may be obliged to pay the costs of supervision and examination.

Provision is also made that landing piers are to be used only to the extent and in the manner for which the permission has been granted. The circular also deals with piers used as promenades or in connection with bathing places, and places them under the supervision of the local police authorities. With regard to existing structures, it is provided that if it is found impossible to keep them in proper repair they must be closed to the public.

The Relations of Consulting and Municipal Engineers in Great Britain*

By H. C. H. Shenton†

There should be no antagonism between the Consulting Engineer and the Municipal Engineer. They are both members of the same profession and for that reason should carefully study their common interest. Signs are not wanting that the time is ripe for combination among engineers with the object of improving their standing and general welfare, and the present paper is written with the object of furthering this end. There is absolutely no difference in status between the two branches of the profession. We are all engineers, we all start level, we all receive much the same education and training, and eventually elect to specialize or to take up municipal appointments or to take up some other branch of the profession not under present consideration.

The combination of all classes of engineers into one body is not beyond hope, but the combination of engineers of a particular class should not only be quite possible, but may well be attempted at the present time. For instance, all engineers holding municipal appointments could combine very easily, but in the author's opinion such combination would be incomplete, because it would leave out a very large number of men who advise us to the construction of some of the most important works of local authorities, viz., the Consulting Engineers.

A difficulty at once presents itself as to who should be included in such an organization and who should be excluded. It is not to be denied that there is a certain number of Consulting Engineers whose experience does not entitle them to call themselves "specialists," and whose knowledge of the subjects upon which they undertake to give advice is inferior to that of the average Municipal Engineer. Any man, however limited his knowledge may be, is at the present time able to write the letters C. E. after his name, take an office and give advice, and if he possesses influential friends and a plausible manner he stands a good chance of obtaining work with which he is quite unqualified to deal. There ought to be some method whereby such persons should be prevented from advising as to the spending of public money. On the other hand (and it is necessary to be perfectly frank), there are many persons holding positions in municipal or District Council Offices who by no stretch of imagination can be considered worthy of the title of "Engineer." In the author's opinion a District Surveyor is, or should be, an Engineer, yet in small urban and rural districts, and even perhaps in some Boroughs, where the salaries paid to the District Surveyors are too small to tempt experienced men to act, men of a very inferior type as regards experience are frequently employed. Ignorant Councillors are frequently of opinion that if a man is a good bricklayer or tradesman he is an engineer, and they employ him as their Surveyor partly from motives of mistaken economy, and largely because they prefer to have someone to whom they can dictate, and whom they will treat with very little respect, thinking that thereby they, being omniscient in all matters pertaining to the Surveyor's Department, may have the ruling hand. In this way the dignity of the Engineers' position is lowered and an enormous waste of public money takes place.

The properly trained and qualified man, whether he is a Municipal or a Consulting Engineer, would like to see the profession so organized that the incompetent person should find it as difficult to call himself an engineer as to call himself a lawyer or a doctor.

However, at this point one must carefully avoid coming to a false conclusion. There are many good engineers whose early training was that of a different calling, but who possessed such ability that they forced themselves forward, gaining experience it may be in a very humble station, and finally taking their places in the profession as honored members with absolute justice. It is, therefore, necessary to consider whether many of the men who cause so much annoyance by undertaking work for which they can show no proper qualifications, and by doing it for an entirely insufficient remuneration, are not often persons of considerable ability whom it would be difficult and unwise to treat as being completely outside our ranks. The very fact that they possess the assurance to undertake engineering work suggests the possession of one of the qualities most necessary to all engineers, viz., pluck.

These men, whatever their previous condition may have been, may be quite capable of gaining the necessary experience and training to qualify them as engineers, but until they have done so, whatever their age and position may be, they can only be regarded as juniors, and should be treated accordingly. In that case the local authority who tried to employ such unqualified persons in chief positions or as consultants should be prevented in the same way as they could be prevented from appointing a medical student to the position of Medical Officer. Leaving out of our consideration the inexperienced class of men on both sides, we may consider the remainder as belonging to one class, and these ought to be able to unite for their common good.

The most important question at the present time is that of remuneration. No engineer ought to be made to work for insufficient payment. The Municipal Engineer is sometimes a perfectly competent specialist, and can advise his Council better than any consultant upon a given matter outside his ordinary duties, but when he does so without receiving proper fees such as the consultant would charge, he is doing the profession serious harm. He is ranking himself with a sort of consultant who prepares schemes for nothing, or who obtains work by offering to do it at a reduced commission. In the abstract one must condemn such practices utterly, but such is the unfortunate condition of the profession, owing however, entirely to lack of organization, that the Municipal Engineer may be forced to do work quite outside his ordinary duties without charge, in order to keep his position, and the consultant may be forced to work for nothing in the hope of being able to make a living some day. This is entirely wrong but it has been done. There is only one remedy: we, as engineers, should combine and organize for registration, so that the incompetent men may be excluded from the ranks of those who act in chief positions, and when this has been done we shall be able to refuse to work except to a proper scale of charges and moreover we ought to be prevented from doing so. Means must be devised whereby it is made impossible for local authorities to obtain the services of their salaried officials or of consultants, except by paying proper salaries or proper fees.

It is urged at times that the Municipal Engineer is unfairly taking up the work of the consultant. This, in the author's opinion, is quite incorrect, for it is obviously the bounden duty of a Municipal Engineer to give his best services to his employers, and so long as he can do the work and his remuneration is adequate, the Consulting Engineer has no grounds for complaint. Where a consultant is called in to advise a local authority his work should be done in such a way as to assist and strengthen the position of the Municipal Engineer, and similarly the Municipal Engineer should work with the consultant so as to insure the success of his work in every possible way, both sides avoiding doing anything likely to bring the profession into disrepute. In the author's experience these relations are generally quite satisfactory, but it would be of considerable interest to learn the experience of others upon this point.

A Consulting Engineer is sometimes called in by an authority whose Surveyor is a man of knowledge and experience, but whose services are entirely under-valued and underpaid by his Council, and whose position is the reverse of pleasant. It is possible in such a case for the consultant to insist upon proper respect being paid to such a man, and he should make it his business to do so and to help him to advancement in every way possible, as the Council are likely to attach importance to the action of the consultant and to the degree of respect with which he treats the Surveyor.

It may be asked whether it is possible after all to make an exact distinction between the man who is an engineer and the man who is not. In the author's opinion there is no difficulty whatever in deciding whether a man is a qualified Municipal Engineer. It is not at the present time a question of examinations passed, of pupilage or of membership of societies and institutions. It is simply and solely a matter of training and experience, and unless a man has been occupied for an extended period on municipal-engineering work he cannot by any possibility be rightly called a Municipal Engineer. A Civil Engineer is not necessarily a Municipal Engineer any more than a doctor is necessarily a medical officer.

Similarly with regard to consultants, the Consulting Engineer is a specialist, and this implies that he has made a special and extended study of some particular branch of engineering, such as water-works, sewerage or electrical work. He is an expert in some particular work, and it is obvious that he is sailing under false colors if he calls himself an expert unless his knowledge and experience of the particular work he undertakes are extensive.

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*Slightly condensed from a paper before the Institution of Municipal Engineers, in London, June 25, 1913; with discussion much condensed from "The Surveyor," of July 4, 1913.

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It is very desirable that engineers in private practice who undertake to do work upon which public money is spent should be carefully registered not simply as engineers, for that term is too vague, but as water engineers, gas engineers, sewerage engineers, tramway engineers, etc., as the case may be. It is after all quite a simple matter for a man who poses, say as a gas engineer, to show that his experience is such as to qualify him to give advice on that subject; if he cannot, he is not fit to advise as to the spending of public money on gas works. Municipal Engineers can help their consulting brethren on this point greatly by urging their authorities to see that their consultants on any particular work are really specialists for that work, and not for something else. They can also assist very materially by joining energetically as a body in any scheme for combination which may be forthcoming for the better organization of the profession.

DISCUSSION

Arthur J. Martin wrote [in part] as follows: The need for protecting the public against incompetent men who pose as engineers is urgent, but the British public is morbidly suspicious of "rings," and in their present temper I fancy they would rather that a hundred local authorities were victimized by unqualified engineers than that one man who by any stretch of imagination might be considered an engineer should be excluded from practice. Any proposals in the direction of registration should therefore be framed on the most moderate lines, and with the utmost consideration for those men of undoubted ability who have not had the advantage of a regular engineering training. The questions of remuneration and conditions of employment are very difficult, and it would be a mistake to try to legislate on them without some consultation with the various associations representing the local authorities who employ us. They are reasonable men for the most part, and should fall in with any regulations framed as much in their interests as our own. But they would certainly resent anything which looked like an attempt to restrict competition or put up fees; and it is sheer waste of time for one side to try to make rules for the game if the other side won't consent to abide by them.

W. Matthews Jones (Chester) said the consulting engineers, as a rule, was caled in by the council, and at once became top dog. He knew nothing, as a rule, of the local conditions. He pumped the municipal engineer to find out all about these, and when he knew all he wanted to know he considered he was the man to carry out the work. He (Mr. Jones) thought some steps should be taken with a view to some sort of combination to stop undesirable men from getting such a position. Recently an architect was employed by a local authority to carry out a sewage scheme for an outside district, at a cost of £8000. He had many friends on the council, and that was why he got the job. A Local Government Board inquiry was held, and the man appointed told the inspector what he had done. The inspector found out that he had been advised by some London engineer, and that the architect could not explain the details of the scheme. The council, of course, were to blame for employing this man, and councils should be stopped from doing this sort of thing.

Horace Cubitt (London) asked if the term "consulting engineer" as used was not a misnomer. A lot could be said nowadays for the existence of capable consultants in any branch of constructional work, for they who were engaged in that work came across special classes of work where they would be glad to have the advice of someone who specialized. The consulting engineer, however, as he appeared in municipal work, rather seemed to go contrary to his name. Instead of acting as consultant, the practice was for him to have charge of and carry out works over the heads of the permanent officials, instead of acting with those officials, and advising them what from his experience was the best thing to do.

He was interested in what the author of the paper said about registration. Architects had been dealing with that matter, and had come to the conclusion that the only way they could get registration was by limiting it as much as possible. The engineer, like the surveyor, seemed to be in rather a worse position, for the term "engineer" was so comprehensive. The surveyor had more trouble in defining his position than the architect, and the engineer had far more trouble in explaining what he was than the surveyor. The man who looked after a gas engine at a factory was an "engineer," and it appeared therefore that engineers in trying to get registration were up against a difficult job. The Government would not interfere with people carrying out constructional work. They might prohibit people from asserting that they belonged to a certain class, but they would not prohibit them from doing any work.

A. H. Dykes (hon. secretary of the Association of Consulting Engineers) thought the last speaker had rather hit the nail on the head when he stated that the term "engineer" was so comprehensive. As Mr. Cubitt had truly said, anyone could call himself an engineer, and there was no likelihood of getting any legislative action which would define what an engineer was. It had been his fate to consider the question of defining one branch of engineering—that of the consulting engineer. The definition of a consulting engineer finally arrived at by the association was that such a person should possess the necessary qualification of one or more of the various branches of engineering, that he should devote himself to advising, that he should have his own office or staff, and that he should not be directly or indirectly affected by commercial interests to such an extent as would affect his judgment in his advisory position. This was designed to provide that the consulting engineer should be so situated as to be able to give impartial advice, and not have commercial ties which might influence his action. It was a fact that the term "consulting engineer" was a very loose one indeed, and that nothing prevented a man from calling himself a consulting engineer. It seemed to the large body of consulting engineers that the time had come that they should band themselves together to form an association by which the public might know that if they applied for advice to the members they would be able to obtain it from men in a position of independence. The association made no attempt to prevent anybody calling themselves consulting engineers; what they relied upon was in briging together a body of men whose qualifications were above reproach, and whom the public and local authorities—and he spoke as a member of a local authority and a member of committees—might appeal to for the right class of man. This was one way in which they were educating the public in this matter.

As regarded the qualification of municipal engineers, that was for the profession itself to decide. The men composing a body were obviously best able to decide what the qualifications of the members should be, just as the members of the Consulting Engineers' Association laid down what were the qualifications of members of their association. There was no difficulty, said the auhor of the paper in deciding whether a man was a qualified engineer. "It is not," he said, "at the present time a question of examinations passed, of pupilage, or of membership of societies and institutions. It is simply and solely a matter of training and experience, and unless a man has been occupied for an extended period on municipal engineering work he cannot by any possibility be rightly called a municipal engineer." But how was an outside body, how was the public to know, and to be able to judge what were the experience and qualifications of engineers? He apprehended that could only be arrived at by taking the views of some recognized association, and that was the purpose intended to be served by the Institution of Municipal Engineers. The Association of Consulting Engineers placed a large amount of power in the hands of the members, and before any member could be admitted his nomination had to be passed by the committee, and afterward confirmed by the full association. It was insisted that a member should be a corporate members of the Institution of Civil Engineers, or of the Institutions of Electrical or Mechanical Engineers, which bodies, it might be stated, were gradually making the qualification for admission stronger and stronger.

Horace Boot (London) remarked that if it were the practice of all engineers in municipal employ to appeal more frequently to a higher authority, or rather to the specialist in a particular branch, as the sagacious town clerk did on every conceivable occasion, the better it would be for the municipal engineer. The town clerk never accepted any responsibility upon his own head. Whenever a difficult question arose with respect to which he might be blamed, he advised the corporation to take counsel's opinion, although he was probably quite as competent as counsel to give an opinion. Being a lawyer he wisely said to himself, "Why should I, receiving a salary, take upon myself a responsibility for which I am not paid?" He had so educated his council that they thought him an exceedingly wise man for the position. It was to be hoped they would see a similar state of things with the municipal engineer, who was undoubtedly a most overworked and underpaid official, and the most knocked about by the council, because, of course, every councillor "knew something about" engineering—being builders and contractors in various ways. The town clerk always managed to get more money for his services than the engineer, for what reason he, as an old municipal official, had never been able to ascertain, seeing that 90 per cent. of the duties of the town clerk could be performed by any ordinary skilled clerk. If the engineer suggested that outside assistance should be obtained there seemed to be an idea that he was not sufficiently competent, though this might not be the case at all. If engineers as a

body could combine to present a solid front councils would in time look up to them as they did at present to the town clerk when that official suggested that counsel's opinion should be taken, and they would be a happier lot, and instead of any hint of antagonism the consulting engineer would be, as he ought to be, the greatest friend of the borough engineer.

The author referred to registration, and one of the speakers stated that the term "engineer" might mean anything, hence the necessity for registration. It appeared to him (the speaker) disgraceful that a fitter in a shop, who might be a good mechanic, should call himself an engineer, and so mislead the public as to his qualifications. It was a standing disgrace to the Institution of Civil Engineers that they had not had the pluck or courage to tackle this problem. It was for them as premier institution to show the way and take steps to prevent the term "engineer" being utilized in the wide sense it was.

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Concreting a Tunnel by Compressed Air

By A. C. Everham*

Probably the largest job in which concrete has been mixed and conveyed by air has been finished recently in Kansas City, Mo., in the building of the O. K. Creek sewer by the Kansas City Terminal Ry. This sewer, which drains some 3718 acres in the heart of the city, ran through an open channel across the site of the new union passenger station. As now diverted it is carried across beneath the front of the new station. From 22nd and Walnut Sts. to 25th and Jefferson Sts. it is in the form of a circular sewer, and thence it is a double box sewer to 25th St. and Fairmount Ave. The total length is 6160 ft.

At about the middle of this sewer is a tunnel 1230 ft. in length, through solid rock and shale ledges. The excavation of the tunnel was carried on at first from a shaft 420 ft. from the west end, and afterwards from the two portals as well as from the shaft. It developed that over the entire length of the tunnel there was a good ledge of rock for a roof, which was nearly self-supporting. The tunnel is lined throughout with concrete, which is about 8 in. thick in the invert and 18 in. at the crown of the arch.

In planning the work, the problem was to find a method of doing it in an economical and expeditious manner. Several plans were suggested for the mixing and conveying of the concrete from the surface of the ground to the forms in the tunnel. Among these was that of boring holes from the surface at frequent intervals and conveying the concrete on tracks above the ground and depositing it through the chutes. The attention of the contractor was called to the fact that a Chicago company had an air mixing and conveying machine, which it installed on a royalty basis, understood to be 50c. per yd. It was decided to experiment with this machine.

In order to demonstrate the efficiency of the process and the character of the concrete which these machines would mix and convey, the contractor was authorized to use a machine on part of the open-cut work, immediately west of the tunnel. This machine was used to carry concrete in the open cut 350 ft., and later concrete was carried in the tunnel 450 ft., with entire success. It was found that it was necessary to install an air-storage tank of about 125 cu.ft. capacity to furnish the necessary quantity of air to drive the concrete through the pipe.

*Assistant Chief Engineer, Kansas City Terminal Ry., Kansas City, Mo.

An air compressor with 90 lb. pressure at the compressor, and with a rated capacity of 650 cu.ft. per sec., had already been installed for the purpose of operating drills and hoisting machinery in the open cut and tunnel, so that no equipment of this kind had to be purchased for the pneumatic mixer.

The air used in the mixing and conveying of the concrete was conveyed by two 2-in. pipes from the compressor to a storage tank, mentioned above, with a drop of 10 lb. in pressure in this line. When the concrete is being deposited there is a still further drop of 20 lb. in the tank and delivery line.

After half of the open-cut work was completed, the machine was used in the tunnel work, and was first put in the shaft, 420 ft. from the west portal, as shown in Fig. 1. This shows the arrangement of the shaft, mixer, pipes and forms in the tunnel.

About 200 ft. west of the shaft the first form was set up, and the tunnel lined from that point to the shaft. After the completion of the lining in this portion of the tunnel, the mixer was taken out and set up at the west portal for use in closing the gap from the open-cut construction to the completed section of the concrete sewer in the tunnel. By the time this closure was made, the excavation for the east two-thirds of the tunnel was completed, and concrete was started with two machines, working from about the center of the east portion of the tunnel. One machine was located in the shaft, and one at the east portal of the tunnel. The diagram (Fig. 2) shows the progress made by the two machines. The best week's record made by one machine was 105 ft. of completed tunnel lining, with two sets of forms, running from 25 to 30 ft. per section.

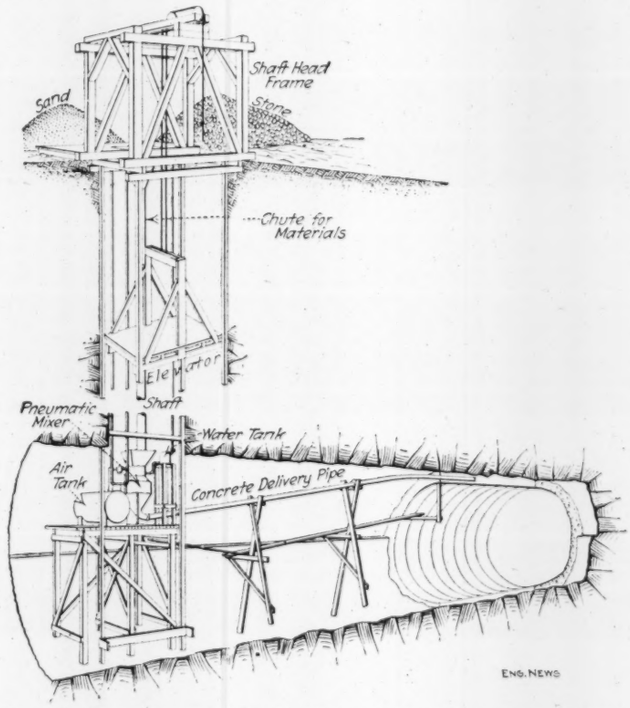

Fig. 1. Lining the O-K Creek Tunnel with Concrete by Means of Compressed Air; Kansas City Terminal Ry.

The mixing machine is shown in Fig. 3. It consists of a charging hopper *A*, which empties into an iron drum *B*, by gravity, the top of the drum having a trap door *D*,

which is held tightly closed when the compressed air is admitted to the drum. The door is operated by the cylinder *D*, supplied with air by the small pipe *E*. Into the drum, the water, sand, cement and stone are dropped in their proper proportion, the cement being between the rock and sand. At the bottom of the drum there is an 8-in. discharge pipe *F*, to which is connected the pipe leading to the place where the concrete is to be deposited.

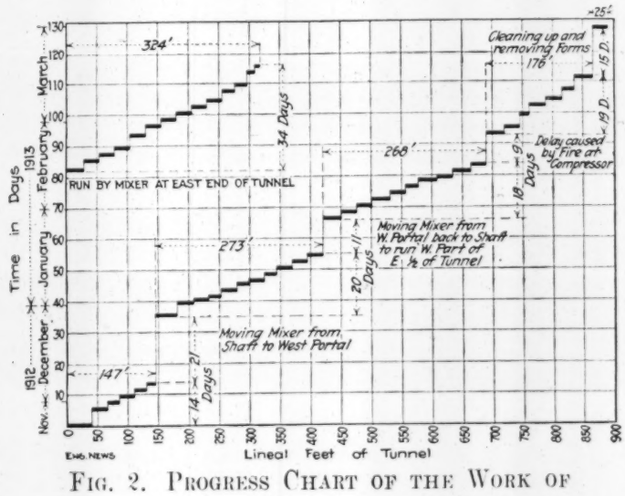

FIG. 2. PROGRESS CHART OF THE WORK OF CONCRETING THE SEWER TUNEL

The air pipe from the storage tank is connected up at the top of the drum, at *G*, and at the bottom of the 2-in. discharge at *H*, so that when the air is turned on, it blows air down through the charge of material and at the same time drives this charge through the delivery pipe. The drawing shows the position of these pipes.

When the air is turned on, it delivers the concrete short distances in a very few seconds, and delivers it at a distance of 350 ft. in 30 sec. The delivery pipe was an 8-in. flange-coupled pipe. The size of batch used was

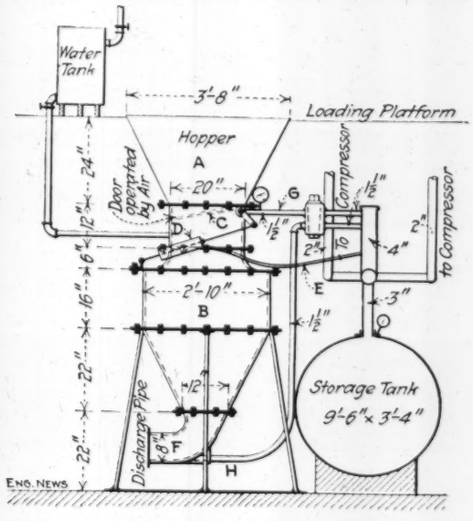

FIG. 3. PNEUMATIC APPARATUS FOR MIXING AND DELIVERING CONCRETE

7 cu.ft. Although the machine is rated at 9 cu.ft., it seems to work better with the smaller charge. The contractor used wheelbarrows to get the material to the mixer, and with this method averaged 8 cu.yd. per hour. He could easily have made greater progress had he used mechanical means for getting the sand and stone to the mixer.

At the point of delivery it is very necessary to have something substantial to take the impact, as the charge comes out of the pipe with great force. In the open-cut work, we built a box, thoroughly braced and tied to keep it from being broken to pieces by the impact. In the tunnel, the concrete was shot directly into the form, which was thoroughly braced to take care of this impact.

The most important question in connection with the installation of the pneumatic mixer was whether the compressed air would actually mix the concrete properly. This was the real surprise in the whole scheme, as the concrete materials are not mechanically mixed prior to being put in the hopper. It was found to be essential that the proper amount of water should be used. Too much water causes the water and cement to go through the pipe first, the sand and rock following. Too little water causes the pipe to clog, and then it may be very difficult to open it up. It did not take long to find this out, and there has been no difficulty in regulating the water since the first batches were run.

The average rate of tunnel lining per day from the beginning of the work was 7 ft., and of actual working days 11.6 ft. per day. The large difference between these figures is due to delays incident to setting up; one delay of nine days being due to a fire which destroyed the power house.

This method of handling is very well adapted to lining tunnels with concrete where compressed air is available. The mixer was invented and developed by J. H. McMichaels, of the Pneumatic Conveying Concrete Mixer Co., 1025 Jackson Boulevard, Chicago, Ill.* The contractor for the construction of this sewer was Bernard Corrigan. The work was under the supervision of John V. Hanna, Chief Engineer of the Kansas City Terminal Ry.

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A Special Rain Gage, with a large storage reservoir, the whole for use in inaccessible places, where records can be taken only at long intervals, has been employed by the U. S. Geological Survey in its studies of the water resources of Hawaii (Water Supply Paper 218). These gages have been used for points at relatively high elevations, inland from shores of the islands forming the Hawaiian group. Many of these gages are located "in the mountains at places remote from all habitation, and as a rule accessible only by difficult or even dangerous journeys on foot." The object was to provide a gage "which could be read at intervals of one or two months, as the amount of rainfall required and the accessibility of the stations permitted." The gage is described as follows in the publication already mentioned:

This type of gage consists of a galvanized-iron container, 8.40 in. diameter and 24 in. high, with a copper receiver 5.94 in. diameter joined to the container by a tapering neck. To prevent loss by evaporation the bottom of the receiver is contracted to an opening of ⅜ in. diameter, to which is attached on the bottom a short tube with an upturned end forming a trap. With this relation between the diameters of receiver and container a factor of 2 is introduced, so that the container holds 48 in. of rainfall when the water in it is 24 in. deep; when the gage is full to overflowing it contains 51.6 in. of rainfall. Other gages of the same type but larger are made by increasing the diameter of the container to 13.28 in. and 18.78 in. which, by using the 5.94-in. receiver, give measuring factors of 5 and 10, respectively. The diameter of the receiver is made 5.94 in. in all the gages, so that 1 in. of rainfall taken by it will weigh 1 lb. This relation is made use of as a check on the measured depth of water in the gage, the observers being supplied with spring balances and required to weigh the gage before and after emptying.

Careful comparisons have been made of this special gage with the standard daily gage, United States Weather Bureau type, and it has been found impossible to discover any loss by evaporation from the former when read at intervals of one or two months.

*This machine was described in our issue of Aug. 10, 1911, together with its work of lining the La Salle St. tunnel under the Chicago River. In that case, the mixer was on the surface, and the delivery pipe had a vertical drop of about 35 ft. down the shaft, followed by a horizontal run varying from 20 to 240 ft.—Editor.

Railway Terminals in Chicago

SYNOPSIS—In most large cities the railway terminal system is composed of a number of units more or less ill-arranged and inter-connected for economical operation as a single system, this being due to the gradual extension and haphazard development of the facilities of individual roads. This situation has reached an acute stage in Chicago, where the city's growth and development are hampered by a zone or belt of railway terminals around the commercial center. In the present article we describe some of the extensive projects which have been under consideration by the city and have been submitted recently by it to an engineer for examination and report.

During the past few months there has been an active controversy over the railway terminal problem in Chicago, following the presentation of plans by the Pennsylvania Co. for the reconstruction of the old Union Station, with a request for the passage of the necessary ordinance for the work. The discussion of this project led to the presentation of alternative projects by other parties, and to a demand that greater consideration should be given to the public or municipal aspect of the question of terminal improvements.

The present status of the problem is that the city (represented by the council committee on terminals) has declined to approve the plans proposed for the reconstruction of the Union Station, or to pass upon the several other terminal projects which have been submitted. Instead, the committee has received permission to employ expert advice, and on July 1 it appointed John F. Wallace as its engineer to examine and report upon the various plans and the general problem of terminal improvements. It is expected that he will be able to report in October, and meanwhile the whole matter remains in abeyance. Some notes on this recent development are given in our editorial columns, and we give below a review of the present conditions and of the several projects suggested as a means of improvement.

Present Terminals

In Fig. 1 are shown the present six passenger stations, while the railway property (in addition to that covered by the terminal projects shown on the map) is indicated by the stippled areas. The map shows how completely the business district is inclosed, and its expansion is restricted especially by the fact that numerous streets are blocked by this railway property. According to a report of the Chicago Plan Commission, there are 39 east-and-west streets between Chicago Ave. and 22d St., but only 17 of these are through streets. Between the lake and Halstead St. there are 24 north-and-south streets, but only four of these extend through from 12th St. to Chicago Ave. The present central business district has an area of about 4¼ sq.mi. and this is densely congested, but the plans for the improvement of the city provide for an area of about 3 sq.mi. The city has a population of about 2,250,000.

The city is served by 25 railways handling passenger and freight traffic, and there are four minor lines handling freight only or doing a switching and transfer business. These 25 railways use six passenger stations, and it may be noted that the distances from the new Northwestern station are as follows: Union station, 0.15 mi.; Grand Central station, 0.50 mi.; LaSalle station, 0.60 mi.; Dearborn station, 1.10 mi.; Illinois Central station, 1.75 mi. There is no street car or elevated line within two blocks of this last station, and there is no direct local transportation route between the several stations. Through passengers are transferred by busses. The railways are distributed among the stations as follows:

A. Union Station (5): 1. Pittsburgh, Fort Wayne & Chicago; 2. Pittsburgh, Cincinnati, Chicago & St. Louis; 3, Chicago, Milwaukee & St. Paul; 4, Chicago, Burlington & Quincy; 5, Chicago & Alton.

B. La Salle Station (5): 6, Lake Shore & Michigan Southern; 7, New York, Chicago & St. Louis; 8, Chicago, Rock Island & Pacific; 9, Chicago & Eastern Illinois*; 10, Chicago, Indiana & Southern.

C. Dearborn Station* (7): 11, Atchison, Topeka & Santa Fé; 12, Chesapeake & Ohio; 13, Chicago, Indianapolis & Louisville; 14, Chicago & Western Indiana; 15, Erie; 16, Grand Trunk; 17, Wabash.

D. Grand Central Station (3): 18, Baltimore & Ohio; 19, Chicago Great Western; 20, Pere Marquette.

E. Illinois Central Station (4): 21, Illinois Central; 22, Cleveland, Cincinnati, Chicago & St. Louis; 23, Michigan Central; 24, Minneapolis, St. Paul & Sault Ste. Marie.

F. Chicago & Northwestern (1): 25, Chicago & Northwestern.

One feature which calls for special mention is the suburban traffic. This is handled mainly by two roads; 52% by the Illinois Central R.R. and 28% by the Chicago & Northwestern Ry. The former road has two downtown stations exclusively for suburban traffic, at Van Buren St. and Randolph St., east of Michigan Ave. The latter road brings all its suburban traffic into its new terminal station at Madison and Canal Sts., but it has been predicted that a branch line will be built on its right-of-way north of the river to a special suburban station at State St. Only about 7½% of the suburban traffic is handled at the Union Station, whose reconstruction is now proposed.

Some interesting statistics of the operating conditions at the present stations have been prepared by Mr. Sattley (whose plan is described below), and from these we have compiled the summary shown in Table I.

TABLE I. TRAFFIC CONDITIONS AT THE CHICAGO RAILWAY TERMINAL STATIONS

Stations	No. of Railways	No. of trains per 24-hr. Main	Sub.	Total	No. of passengers per 24-hr. Main	Sub.	Total
1. Union	5	222	66	288	13,100	12,900	26,000
2. La Salle	5	98	121	219	5,700	14,100	19,800
3. Dearborn	7	99	33	132	5,800	6,200	12,000
4. Grand Central	4	26	4	30	1,500	1,200	2,700
5. Ill. Central R.R.	4	102	386	488	6,000	90,000	96,000
6. C. & N. W. Ry.	1	77	250	327	4,500	48,500	53,000
Total	26	624	860	1484	36,600	172,900	209,500
		"Rush-Hour" Service			"Rush-Hour" Service		
Total, 7 to 9 a.m.		110	157	267	6,455	31,226	37,681
Total, 5 to 7 p.m.		87	164	251	5,104	32,353	37,457
Total for four rush hours		197	321	518	11,559	63,579	75,138

New Terminals

There are now before the city authorities five plans for new railway terminals. That of the Pennsylvania Co. is proposed for immediate construction upon approval by the city. The others are in the nature of general projects, and have as yet no active backing. All of these terminals are proposed to be located on ground already occupied largely by the railways, although the Pennsyl-

*After Aug. 1, the Chicago & Eastern Illinois Ry. will use the Dearborn Station.

vania Co. proposes a separate freight terminal in a territory not invaded hitherto by the railways. This is one source of objection to the company's plans. A very large

TABLE II. CHICAGO RAILWAY TERMINAL PROJECTS

Project	Character	Station Tracks	Traffic
1. Pennsylvania	1 sta. for 5 roads	Stub end	Passenger and frt.
2. Chicago plan	3 sta. for 25 roads	Stub, loop and through	Passenger
3. Sattley	1 sta. for 25 roads	Through	Passenger and frt.
4. Hunt	1 sta. for 25 roads	Loop	Passenger and frt
5. Pond & Pond	4 sta. for 25 roads	Loop	Passenger

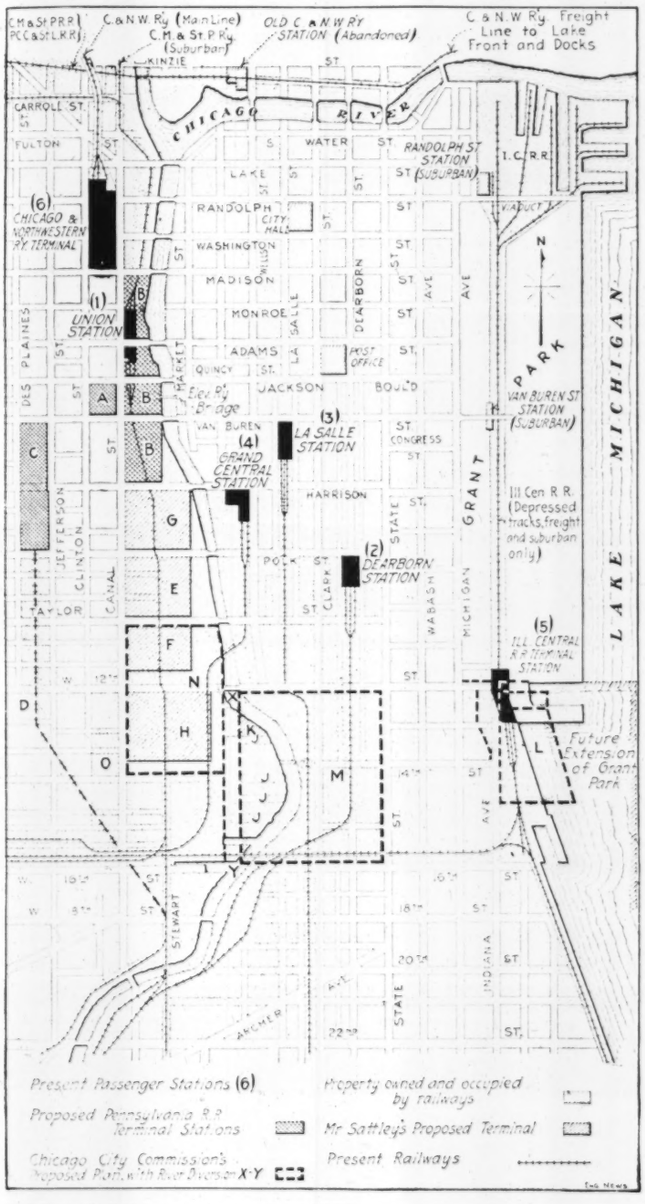


FIG. 1. PLAN OF PRESENT RAILWAY TERMINALS AT CHICAGO, AND THREE PROPOSED TERMINAL PROJECTS

(The stippled area represents land owned and occupied by railways, in addition to that covered by the terminal projects.

Pennsylvania R.R. Proposed Stations

A. Passenger station building.
B. Tracks and platforms (below streets).
C. Freight station.
D. Elevated line to new freight station.

Mr. Sattley's Proposed Union Station

E. Main-line passenger station.
F. Suburban passenger station.
G. Express, baggage and mail building.
H. Freight station.
J. Slips.
K. Power plant.

Chicago City Commission's Proposed Group of Union Stations

L. Station for roads using present Illinois Central R.R. Station.
M. Station for roads using present Dearborn, La Salle and Grand Central Stations.
N. Station for roads using present Union Station.
O. Freight terminal for M., St. P. & S. S. M. Ry. (now being built).

freight terminal is already under construction for the Minneapolis, St. Paul & Sault Ste. Marie Ry. (Canadian Pacific System). This occupies the area between Canal, Clinton, 12th and 15th Sts., and is shown at (O) in Fig. 1.

Some of the general characteristics of the several projects are shown in Table II, and it will be noted that several of these provide loop tracks for the purpose of avoiding reverse movements of traffic. The Pennsylvania Co. project shows the rather curious feature of two groups of stub tracks, separated by the passenger concourse. This is for the reason that it is really a double station, entered at each end by different roads. But the existing station has through main tracks, which plan is adopted also in the Sattley project. We may note that Brussels, which has two stub-end stations at opposite sides of the city, is building a four-track tunnel which will convert these into through stations and thus facilitate the handling of through trains.

THE PENNSYLVANIA PROJECT

PASSENGER STATION—The present Union Station is owned by the Pennsylvania Co., and the company's project naturally provides for the reconstruction of the station on the present site (but covering a much larger area), as will be seen in Fig. 1. The station building proper (*A*) is to occupy an entire block (about 400x320 ft.) east of Canal St., and as all tracks and platforms would be west of this street (*B*) there would be broad subways to conect with the main concourse (180x250 ft.), which lies between Jackson Blvd. and Adams St. There would be no columns on the concourse or platforms. Numerous approaches provide against crowds and congestion of traffic, even during the rush hours, but one objection urged is that the plan does not provide sufficient street widening. The plans include the mechanical handling of baggage, mail and express matter. The cost of this passenger terminal is estimated at $35,000,000. The Union Station Co. has been incorporated to build and operate it; the capital is $50,000,000, of which 50% is owned by the Chicago, Burlington & Quincy R.R., and 25% each by the Pennsylvania and the Chicago, Milwaukee & St. Paul.

In effect, the station would consist of two entirely separate groups of stub-tracks, separated by the concourse, one group for the two railways approaching from the north and the other for the three railways approaching from the south. The former would have ten tracks (in pairs), with platforms 1075 to 1330 ft. long, and a four-track approach to the present line running west on Carroll St. The latter group would have 11 tracks (in pairs), with platforms 1100 to 1380 ft. long and a six-track approach.

In addition to these passenger tracks, the plan provides for a group of express and mail tracks east of the southern passenger station (and between Harrison and Van Buren Sts.). These would be through tracks, serving all the railways. East of these again would be a pair of through tracks permitting of through movements between the two groups of railways. In the present station most of the tracks are through tracks, the concourse being parallel with them. The track plan for the new station is shown diagrammatically in Fig. 2.

The tracks would be at an elevation a little above the river, or 4 ft. below the level of the present tracks in

the station (and 12 ft. below street level). South of the station they would rise to the present level (still below the street level) at Polk St., by a grade of 0.3% from Van Buren St. At 12th St., however, the new approach would rise from the present level by a grade of 0.9 and 1.2% to cross above 18th St., and then descend on a grade of 0.5% to meet the present track level at 23d St. This would eliminate the present complicated grade crossings on the south bank of the river at Stewart Ave. (see Fig. 1). North of the station, the approach would be at the present elevation, which is at the street level westward from Canal St.

FREIGHT STATION—For its own local freight business the Pennsylvania Co. proposes to build a freight terminal station (*C*) at some distance from its passenger station and with an elevated approach (*D*) leaving the main line at 18th St. The station and team yard would occupy the entire space between Jefferson and Des Plaines Sts., from Van Buren St. south to Polk St., and strong objection has been made to this, on the ground that it would seriously interfere with the westward extension of the commercial part of the city.

The freight station (extending to Harrison St.) would have tracks at the ground level and on two upper floors, the approaches to the normal elevation above Polk St. being by ascending grades of 3.85% and 2.1% from the two lower floors, and a descending grade of 0.6% from the top floor. The preliminary plans provide for 12 tracks on each floor. The driveways of the team yard would have 2% grades, and loaded wagons from the inbound freight house (top floor) would descend to the street by loop roadways of 4% grade.

THE SATTLEY PLAN

The plan proposed by Mr. Sattley, Valuation Engineer of the Chicago, Rock Island & Pacific Ry. (and originally suggested by him in 1901),* includes both passenger and freight terminals. The site is on the west bank of the river, extending west to Canal St., and between Harrison and 14th Sts., covering ground already largely owned and occupied by the railways. It would release about 126 acres of the land on the east side of the river (between Van Buren and 16th Sts.), which is now owned by the railways and occupied in an unremunerative manner, but which land is badly needed for the proper commercial development of the city, as explained above. This location also avoids all interference with train service due to drawbridges. Both the passenger and freight stations could be built in units or sections, without interfering with existing traffic. The operating conditions and the means of handling passengers, baggage, mail, express and freight, have been worked out in considerable detail.

The station is designed as a substitute for the present six passenger terminals, except that it would leave the suburban service of the Illinois Central R.R. and Chicago & Northwestern Ry. to be handled as at present. These two roads together handle 80% of the suburban traffic, as noted above, and 7.5% is handled at the old union station, which is adjacent to the proposed terminal station.

PASSENGER STATION—This proposed terminal has four main elements, arranged in a row along Canal St. The passenger station is divided into two separate sections, fronting on opposite sides of Taylor St. The north portion (856x750 ft.) is for main-line traffic (*E*), including transfer from one road to another. The south portion (856x575 ft.) is for suburban traffic (*F*). The former extends to Polk St., and the block from Polk St. to Harrison St. is occupied by a building (*G*) 856x700 ft. for mail, baggage and express. These three streets would be 150 ft. wide. The headhouses of the two passenger stations would have several floors for office purposes, and basement floors with tunnel or subway connections to the center of the city. The baggage building also would have several floors for the storage of baggage and express matter that is not to be distributed immediately. Its basement floors would connect with special subway lines to downtown offices or collecting and distributing points, thus reducing the team traffic on the streets, while the use of a single station would eliminate the great amount of team traffic involved in the transfer of through baggage between several stations.

For the 25 railways, the station would provide 30 through tracks, each about 1675 ft. long, or of such length as to be available for two (or more) trains entering or leaving at opposite ends of the station. These would be in pairs, separated by 30-ft. platforms. In addition there are 15 tracks for holding empty trains which accumulate from a rapid inbound suburban service or which are held to furnish a rapid outbound service during the rush hour. For baggage and mail, 15 through tracks are provided, connected at each end with the running tracks, and served by platforms 15 ft. wide and 1000 ft. long between the pairs of tracks. It is estimated that the track system would provide for handling 4320 trains daily (at 5-min. intervals). Inbound trains entering at either end would proceed (when empty) in the same direction and run to coach yards and engine facilities beyond the terminal, utilizing existing yards and railway property to a large extent. Outbound trains made up in the yards would pull into their proper places in the station, and pull out in the same direction when loaded. In this way reverse movements would be eliminated. All tracks would be at an elevation about 6 ft. above the level of water in the river, and 22 ft. below the streets and the main floors of the stations.

For transportation between the terminal and the business section of the city a four-track subway loop and a four-track elevated loop are proposed, forming part of the local transportation systems and crossing the entire width of the station. These would be in addition to the street-car service. Provision is made also for tunnels for handling baggage, freight and mail between the terminal and certain downtown points. Several broad streets would cross the terminal site, and two new bridges are suggested (at 14th St. and at the location of the present railway bridge south of Taylor St.) New 80-ft. wide approach streets on the east side are proposed, on the property now occupied by the railways.

FREIGHT STATION—The freight terminal (*H*) is located south of 12th St., a space of 400 ft. wide separating it from the suburban station, so as to provide ample space for movements of teams and general traffic. The combined freight house and warehouse is a building 1200x1050 ft., several stories high, and with basement floors also, having subway connections for transferring freight between the different units. The basements would connect also

*"Engineering News," May 30, 1901, and Feb. 16, 1905.

with freight-service tunnels for transfer between the terminal and various points in the city, probably in connection with the existing freight-tunnel system which has some 60 miles of narrow-gage line operated by electric motors.

It is proposed to have the building divided into 11 distinct and parallel units for use by the different roads. Each unit would be about 109x1050 ft., with its own yard, and its own driveways on both the street and track levels. The numerous existing one-story freight houses cover about 20 acres in all. The new building would cover about the same space, but (with its several floors) would give 20 acres to each unit, or 220 acres in all. Each unit has track capacity for 858 cars, and it is estimated that a car can be unloaded, removed and another car set in its place in two hours. There is space for 2300 teams at the platforms, and for 3500 teams standing or awaiting their turn.

South of this freight station would be a yard of 800 cars capacity, in which would be placed cars unloaded at one freight house and thus made available for loading at any other freight house. Along the river several slips (*J*) are provided for water-borne traffic with the idea that much of the local freight could be handled by lighterage, thus relieving the streets of a portion of the team traffic. The slips would have elevators serving the subways at the basement level. One of the slips serves the building provided for the heat, light and power plant (*K*).

The Chicago Plan Commission's Project

This commission was appointed by the mayor in 1908 to consider carrying out the scheme for the city's development embodied in the "Chicago Plan." When the present terminal controversy arose, the commission prepared a plan providing for taking care of all the railways in three separate stations. To some extent it is based upon the plan suggested in 1905 by F. A. Delano, President of the Wabash R.R. His plan was for a row of union stations at 12th St., near the river, forming a multiple-unit terminal as distinguished from one immense terminal for all the railways. (Engineering News, Feb. 16, 1905.)

The Commission's plan is to put two stations at 12th St. and the river (*M* and *N*), and a third (*L*) at 12th St. and the lake. This arrangement is not very systematic but is adopted presumably for the reason that the city has decided to make a main thoroughfare of 12th St. and to extend it to the lake, while the Illinois Central R.R. is committed to the reconstruction of its present station on the lake front. By agreement between the city and the railway, the latter is to pull down its present station (which blocks the 12th St. extension) and build a new one at Indiana Ave. and 12th St., as described in Engineering News, Sept. 12, 1912.

The Commission's plan shows this lake-front station (L, Fig. 1) as a dead-end main line terminal. The through suburban tracks would pass beside the station to connect with the present suburban line through Grant Park. The Chicago River would be straightened from 12th to 16th Sts., and two large stations built on opposite sides of the new channel. The eastern station (M), fronting on the south side of 12th St., would be for the railways now using the Dearborn, LaSalle and Grand Central stations. It would have 40 tracks, some being stub tracks while the others would be connected by a loop at the north end of the station. The approach would have six main tracks. The western station (N), fronting on both sides of 12th St., would be for the railways now using the old Union Station. This would have 28 tracks, with a six-track approach from the south and a four-track approach from the north. The station tracks would include both stub and through tracks, while a loop carried above the through tracks provides for transferring trains (of the southern group of railways) from the inbound to the outbound tracks. The track plans of these two stations are shown diagrammatically in Fig. 2.

No plans have been submitted for a freight terminal. It is suggested, however, that this might extend as far north as Polk St., but with all tracks underground so as to permit this territory to be utilized for streets and buildings.

The Hunt Project

This is a very comprehensive plan for a vast union terminal for both passenger and freight service, to accommodate all the railways. It is proposed by Jarvis Hunt, of Chicago, who is architect of the new Kansas City union station, and was noted in Engineering News of Feb. 27. It involves the diversion of the river

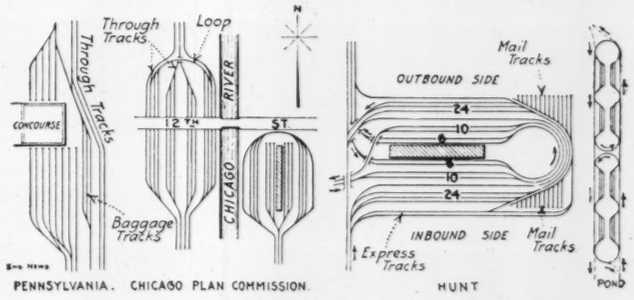


Fig. 2. Diagram of Track Plans of Three Railway Terminal Projects for Chicago

for a considerable distance, and (like Mr. Sattley's project) would necessitate the coöperation of all the railways in one scheme and one station. It is claimed that while it would occupy only a part of the area now occupied by the railways it would provide for six times the present passenger and freight traffic. The cost is estimated at $200,000,000. This project is shown in Fig. 3.

Passenger Terminal—The passenger station would occupy the area bounded by 12th, 16th, State and Wells Sts. Parallel with the new river channel would be the approach tracks, crossing the river by drawbridges near Van Buren St. and 16th St. to connect with the northern and western lines, respectively. These two drawbridge connections form a serious objection to the plan. The drawbridge at the entrance to the old Chicago & Northwestern Ry. station at Kinzie and Wells Sts. was a source of trouble, although this crossed the north branch of the river which has comparatively small traffic. The two bridges in the Hunt plan, however, would cross the main channel of the river, so that the interruption to train service would be more frequent, and in addition they would affect the trains of numerous roads. The drawbridges are an inconvenience to street traffic, but the delays would be much more serious to trains.

The station tracks are at right angles to the approach line (as in the present terminal station at St. Louis,

Mo.) and are arranged in two parallel groups, as shown diagrammatically in Fig. 2, the southern group being for inbound and the northern for outbound trains. Each group would have 10 tracks: 24 for roads entering from the south, 10 for roads from the west, and 6 for roads from the north. The platform tracks are long enough for two trains each. All these tracks could be connected by concentric loops in (or beneath) a plaza at the east

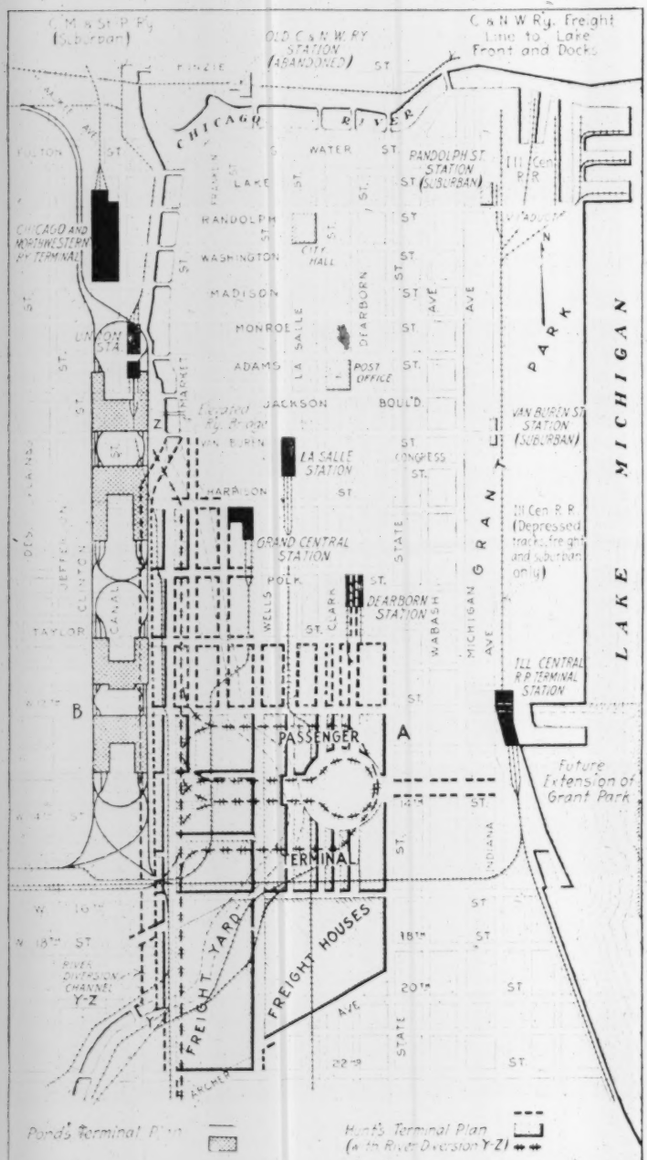


FIG. 3. PLAN OF TWO RAILWAY TERMINAL PROJECTS FOR CHICAGO.

A. Hunt's project.
B. Pond & Pond's project.

end of the station, and their connections to the approach line are so planned as to avoid all grade crossings. It is stated that 600 trains an hour could be handled in this station.

The scheme of operation is that inbound trains would turn from the approach line into the station, and after discharging passengers, would pull around the curve and through the opposite side of the station to reënter the approach and proceed to the coach yards of the individual roads. In the same way, outbound trains (made up in the yards) would run into the station, take passengers, and proceed to the main line. Suburban trains having discharged passengers on the inbound side would stop to take passengers on the outbound side. But (as in Mr. Sattley's plan) it is proposed to leave the bulk of the suburban traffic to be handled as at present by the Illinois Central and Chicago & Northwestern railways. Additional facilities could be furnished, however, by through connections between these and all other suburban lines.

By the proposed track arrangement all reverse movements would be avoided, but it is doubtful whether railway offices would regard with favor the number and extent of the curves involved. All trains entering (except those from the west) would have to describe a curve of 90°, then pass around the 180° loop (with reverse curve approaches from some of the station tracks), and then describe another 90° curve in reëntering the main track.

FREIGHT TERMINAL.—The freight terminal is designed to cover the area from 16th St. to Archer Ave. and from State St. to the river. This would have rows of freight houses on the eastern portion, and on the western portion open team yards with tracks connecting with an underground freight terminal beneath the western portion of the passenger terminal. An objection urged against this is that it would cause great congestion of street traffic, but the designer claims that by his proposed arrangement of double-deck streets there would be no fear of such congestion. It is stated also that the freight houses would provide 20 miles of platforms for teams, while the present numerous but scattered freight houses provide only 3½ miles of platform length. The freight houses would have the driveways at the street level (avoiding all approach grades), with the inbound tracks above and the outbound tracks below. The team tracks and driveways would be at the street level. The new river channel would be 175 ft. from Canal St. and along its eastern bank would be a row of warehouses having water, rail and street connections.

THE POND PROJECT

The plan proposed by Pond & Pond, architects, of Chicago, is for a passenger terminal only. As shown in Fig. 3 it provides for a group of stations to be used by different roads, but connected and coördinated on the multiple-unit system. The location is on the west side of the river, extending west to Clinton St. and from Monroe St. south to 11th St., occupying present railway property. The plan shows four separate stations, placed in a row and fronting on two plazas at Van Buren St. and 12th St. These could be built as units, as required. It is suggested also that the new Chicago & Northwestern passenger terminal (whose tracks are elevated) could be converted easily into a through station connected with the proposed terminal and forming an additional unit of the latter.

Lines of through tracks extend over the entire length of each side of the terminal (for northbound trains on the east side and southbound trains on the west side), with loop connections and with approaches connecting with the several railways so that trains can run to any station and then proceed in the same or the reverse direction to coach yards beyond the terminal. The track plan (Fig. 2) provides separate parallel groups of inbound and outbound tracks, connected by loops as in the Hunt plan, but with loops at each station unit instead of one

station and one loop.* Each station has 28 tracks in two groups connected by loops, as shown, and there are no stub tracks. An inbound train of any road would run directly into the inbound side of its station, and when empty it would pull out around the proper loop to the outbound side to take passengers (if a suburban train) or to proceed to the coach yard of its own line. The transfer of through passengers, baggage, express, etc., from one station to another could be conveniently effected.

All tracks would be above the street level, and a double-deck arrangement is proposed, with main-line traffic on the upper level and suburban traffic on the lower level. There would be no blocking of the streets (except by bridges), and the area occupied by the terminal could be covered by buildings. As to suburban and local traffic, the plan would provide for through routing between the existing lines and for connecting all suburban lines with the main-line stations. Provision is made for an interstation subway loop around the area covered by the terminal, connecting with east and west subway lines extending to loops under Grant Park.

The designers recommend that this terminal should be for passenger service exclusively. They recommend further that no freight terminal should be permitted west of the river north of 12th St., as this would interfere with the westward expansion of the city, which should be facilitated as far as possible to provide for present and future requirements.

*The loops in the new Grand Central Station, at New York, are to be used only by suburban trains (on one level) in passing from the inbound to the outbound side, and by express and mail cars (on another level). The radii of the loop curves for suburban traffic range from 135 to 260 and 400 ft. The radii of the loop curves for express cars range from 330 to 500 ft. at different points on the loops. These curves turn an angle of 180°. Neither of these loops are as yet in service. We are informed officially that while portions of the trains will stand on the loops, no portion of the equipment used by passengers will stand on the loops while the equipment is being loaded or unloaded. The use of these loops is to approximate the conditions of a through-station (so far as possible) and keep traffic moving in one direction.

The International Road Congress Excursion to North Wales

By E. L. Corthell

My promise to supplement the account of the International Road Congress by a brief description of the excursion to North Wales can now be fulfilled, as we returned to London on the evening of July 4 in time for the banquet at the Savoy Hotel to celebrate the great day of the year for all Americans in foreign parts. Mr. S. B. Williamson (late engineer of the Pacific Division of the Panama Canal and now with J. G. White & Co., of London) and myself had the privilege of sitting down with 250 Americans and some distinguished English guests to celebrate our Independence day.

The North Wales excursion was from Chester via Llandudno, Conway, Paenmaenmawr, Carnarvon, Bettws-y-Coed and Llangollen to Shrewsbury. It was participated in by 44 Congress members representing seven different countries and we traveled in three large motor "char-a-bancs." The social functions—lunches, dinners and visits to noted places—were an agreeable diversion from the technical work of road inspection.

"Rare old Chester" is as rare as ever. As all American tourists landing at Liverpool immediately go to Chester it is unnecessary to describe it here, but in this instance as the guests of the City Council and the Mayor, we had unusual facilities for seeing the historic landmarks of this ancient city. After the reception at the City Hall, we spent two or three hours inspecting the well built and maintained roads around Chester accompanied by the Mayor and other city officials and the Engineer, Mr. H. F. Bull.

On the morning of the succeeding day, July 1, we left in our three large motor cars for Llandudno, visiting Hawarden, the home of Gladstone, *en route*. All who know Llandudno, the famous Welsh seaside resort, will appreciate the pleasure we had in seeing it again after an interval of 22 years. We were entertained at dinner at the Imperial Hotel by the Chairman and Urban District Council.

Resuming our tour next day, we stopped at Conway, where the Mayor and the City Engineer showed us the famous old castle. At the Penmaenmawr Welsh Granite Company's quarry we stopped to see one of the largest quarries of granite in Great Britain. We were conducted over the quarry on half a dozen cable inclines by the President, manager and engineers. A heavy blast was set off for us on one of the upper terraces.

We arrived that night at Carnarvon where a dinner was given us by the Carnarvon County Council and in the morning we had an opportunity to examine the Castle, one of the best preserved feudal castles in the British Isles. It was the birthplace of the first Prince of Wales. King Edward First was in dispute and nearly at war with Wales, when in a conference he agreed that the first prince of his family should be born in this Welsh Castle. This excellent stroke of diplomacy settled the business satisfactorily and the prince was born there shortly afterward.

On this day's tour, we left the Welsh coast, which we had skirted all the way from Chester and turned inland through the Welsh Mountains, penetrating the Vale of Llanberis in the shadow of Snowdon, the highest mountain peak in Wales, where enormous slate quarries were passed, and lunching at beautiful Bettwys-y-Coed. The location of this road through a difficult mountain region makes one honor the ability of the engineers who constructed it before the railroad era. In fact we traveled nearly all day over the Holyhead and London road, 259 miles long, built by Thomas Telford. While the surface of this famous old road has been many times renewed, the firm foundation laid under Telford's direction is still doing good service.

We spent that night in the interesting Welsh town of Llangollen and reached Shrewsbury on the following day.

At Shrewsbury the party was received by the Mayor and Council in the Guildhall, where we were offered the "loving cup," a custom going back a hundred years at the official reception of those whom the city wishes especially to honor.

It goes without saying that we all partook of the contents of this great silver tankard, rising and making our best bow to the Mayor, who sat in state with his gorgeous costume and insignia of office and his mace and sword bearer behind him.

A luncheon in the Music Hall by invitation of the Association of bituminous and other paving material manufacturers completed the excursion and the party returned to London by the London & North Western Ry.

Sewage Disposal at New Bedford, Mass.

By Walter N. Charles*

SYNOPSIS—New Bedford is building an intercepting sewer, with screen and grit chambers at its lower end, and an outfall sewer into Buzzard's Bay, about 3300 ft. long, extending to 30 ft. of water. The sand removed from the grit chambers will be cleansed by washing before it is discharged onto the adjacent beach. Provision is made for adding apparatus to disinfect the sewage at the screen and grit chambers should that be considered necessary in the future.

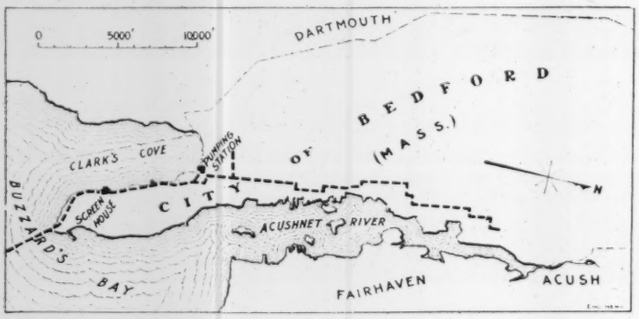


Fig. 1. Location of Intercepting and Outfall Sewers, New Bedford, Mass.

The city of New Bedford is located on two arms of Buzzard's Bay: Clarks Cove on one side and the so called Acushnet River on the other. Both are comparatively narrow and shallow bodies of water. At present all the sewage of the city is emptied directly into the cove or the river at the shore line by 39 outlets. The range of tide in these waters is only about 4 ft. and consequently the diffusion of sewage is very slow. This condition of affairs has resulted, after many years, in a very obnoxious pollution.

Besides the generally bad condition resulting to the city on account of the present sewage disposal, there is the danger of infection of sea food, as the taking of shell-fish is becoming a considerable industry in this vicinity. The beaches also are so filthy that bathing is unsanitary and dangerous to health.

At present the city has about 100 miles of sewers, mostly on the combined system. To take care of these and to provide for future extensions, an intercepting sewer (Fig. 1) has been designed to run through the city in a southerly direction close to the shore line where the topography is low and fairly flat. This will be a gravity sewer and will discharge in Buzzard's Bay some 3300 ft. beyond the extreme southerly shore line in water about 30 ft. deep.

The point of discharge was determined upon as a safe distance from land to provide for a proper dilution and diffusion of the sewage in plenty of pure water. A series of tests with submerged floats at various depths showed that the possibility of any sewage reaching land from this outlet was very remote, the prevailing winds and currents being favorable to good dispersion.

There are certain small areas of the city which are too low to drain into the interceptor directly so that it will be necessary to build three or four small pumping stations to force the sewage up to a proper grade. These stations will all be automatic with electrically operated pumps or air-compression systems.

*Assistant Engineer, New Bedford, Mass.

Intercepting Sewer and Overflows

The intercepting sewer was designed to take care of 120,000,000 gal. per day on a 0.03% grade. This capacity will be sufficient to take all the sewage for a population of 300,000, together with 1 in. of rainfall per day for one-third the area of the city. The remainder of the surface drainage will be carried by direct outlets or by overflow pipes from the branch sewers controlled by regulators. These regulators will be located at the junction with the interceptor and will operate so as to overflow the excess storm water into the present outlets to the bay by inverted siphons under the intercepting sewer (see Fig. 2). The amount of water carried by these overflows will be small, probably not more than one-fourth of the surface drainage, and that will come only in severe and continued storms so that it will be only slightly polluted by sewage.

The main interceptor is about seven miles long, and

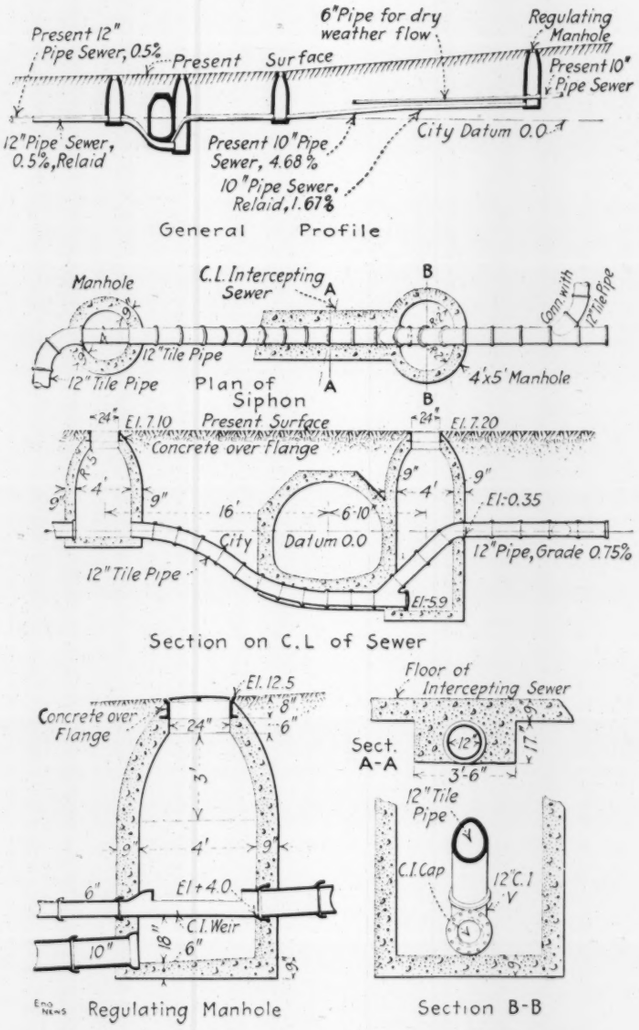


Fig. 2. Storm-Water Overflow, New Bedford Intercepting Sewer

is being built of reinforced concrete, 1:2:4 mixture. The upper end is to be equivalent to a 3-ft. diameter circular section and will be increased as necessary to a 7x7⅔-ft. horseshoe section at the extreme end of the land (see Fig. 3). There is also a branch intercepting sewer of 48x50-in. section for about one-third of a mile across the city near the south end. Blaw collapsible forms are being used in the work already started.

On the large reinforced-concrete sewer in open cut from 10 to 24 ft. depth, accepted bids range as follows:

Earth excavation	$1.25 to $1.50 per cu.yd.
Rock excavation	4.00 to 5.75 per cu.yd.
Concrete masonry	4.75 to 9.70 per cu.yd.
Steel reinforcement	0.016 to 0.02 per lb.
Underdrain, 6 in. (pipe furnished by city)	0.15 to 0.30 per lin.ft.
Sheeting in place	20.00 per M. ft. B.M.

Screen House and Grit Chamber

As the intercepting sewer system is obliged to carry a large amount of surface drainage, it was considered necessary to devise some means of removing the sand before it reached the outfall sewer, where it would no doubt settle and clog up the outlet in a short time. To remove this sand and also to eliminate objectionable floating material which might be brought with the sewage, a screen and grit chamber has been designed.

The building, 65x85 ft. in plan, will be erected over three sewage channels, the central one to take the dry-weather flow, and one on either side to take the storm flow (see Fig. 4, p. 218). All channels are provided with screens and gates and, if necessary, may be shut off at any time.

The screens are box shaped, with vertical steel bars ¾ in. in diameter spaced 1½ in. c. to c. around three sides, the fourth side, or front, being left open toward the flow (see view Fig. 5). The bottom of the box is of steel plate with holes for drainage. The top is also of steel plate and there is a framework of steel connecting the top and bottom and supporting the rods. The sludge collected in the screens will be burned in the power house adjoining the chamber.

The two storm channels have large settling basins 50 ft. long with floors 6 ft. below the sewer invert where the sudden check in velocity will cause the sand to drop. One channel only will be used at a time, the other being cleaned meanwhile. The chambers are large enough so that no baffle-plates or other mechanical means of stopping the sand will be necessary.

To clean the settling basins, sand pumps will be installed. These pumps will force the sand directly to washers, which will be located over the central dry-weather flow channel and will be similar to the sand washers used at water-filtration beds. After the sand is washed it will be run out for filling on the shore. In some localities this sand washing would be unnecessary but the screen house is located in a residential district and the odor or contamination from sewage-laden sand would be very objectionable.

Should the city in future years find the pollution of Buzzard's Bay to be increasing to an unreasonable degree, the sewage will be treated with disinfectants, and to provide for this emergency a mixing room is to be built in the screen house where a solution of bleaching powder can be injected into the sewage through various small openings for the purpose of destroying the dangerous bacteria.

Junction Chamber

The reinforced-concrete sewer meets the submerged outfall in a junction chamber, in which there is a hydraulically operated sluice gate on each gate of the two lines of 60-in. pipe. Just back of this chamber is a tide-gate chamber with two tide gates, one behind the other, two being placed to insure a tight closing. The sluice gates are for emergency and flushing purposes only, but the tide gates will operate continuously to prevent high tides from backing up in the sewer. Although high tide could back the sewage up for about two miles, it is not probable that the sewer will ever be under head except at the extreme end of the land section and in the outfall.

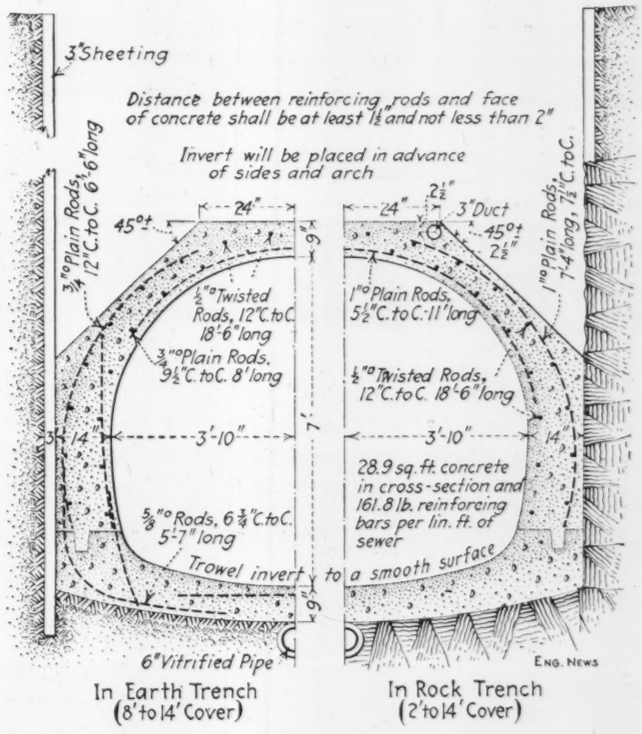

3" Sheeting
Distance between reinforcing rods and face of concrete shall be at least 1½" and not less than 2"
Invert will be placed in advance of sides and arch
¾"ø Plain Rods, 12"C. to C. 6'-6" long
45°±
24"
9"
½"ø Twisted Rods, 12"C. to C. 18'-6" long
¾"ø Plain Rods, 9½"C. to C. 8' long
14"
3'-10"
7'
5/8"ø Rods, 6¾"C. to C. 5'-7" long
Trowel invert to a smooth surface
9"
6" Vitrified Pipe
In Earth Trench (8' to 14' Cover)
2½"
3" Duct
45°±
2½"
1"ø Plain Rods, 7'-4" long, 7½"C. to C.
1"ø Plain Rods, 5½"C. to C. 11' long
½"ø Twisted Rods, 12"C. to C. 18'-6" long
28.9 sq. ft. concrete in cross-section and 161.8 lb. reinforcing bars per lin. ft. of sewer
In Rock Trench (2' to 14' Cover)
ENG. NEWS

Fig. 3. Typical Sections of Reinforced-Concrete Intercepting Sewer, New Bedford, Mass.

(As designed for Fort Rodman Reservation and West French Ave. In earth trench with 2 ft. of cover the design showed 27.3 sq.ft. of concrete in cross-section; the reinforcing bars showed 84 lb. per lin.ft. of sewer for 2 to 5 ft. of cover and 88.65 ft. for 5 to 8 ft. of cover. The design also provided that if soft ground were encountered, the side walls might have to be thickened and more reinforcement provided, as shown at "A," "B" and "C" on the section for earth. Throughout, the invert was to be laid first.)

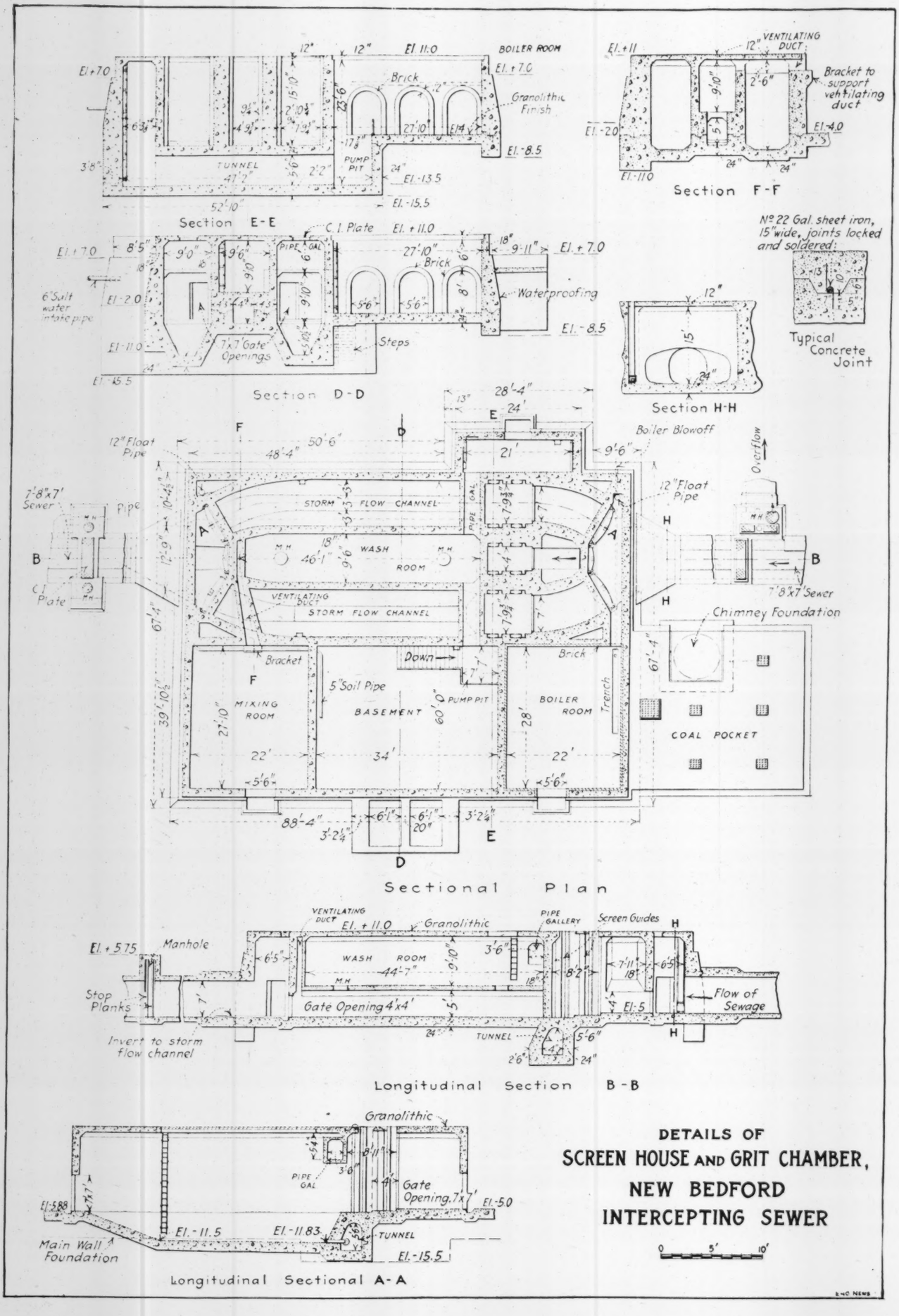
The Submerged Outfall Sewer

The outfall, laid in a dredged trench under the bay for 3288 ft., will eventually consist of two lines of 60-in. cast-iron pipe, only one of which is now being laid. The other line will be laid when required by increase in population.

The contract for the outfall was awarded to Patrick McGovern at the following prices (the pipe being furnished by the city):

26,000 cu.yd. excavation @ 55c.	$14,300
3,288 lin.ft. 60-in. pipe laid and backfilled @ $16	52,608
Placing and riprapping outlet	2,000
Total	$68,908

The average depth of the trench was 8 ft., mostly in hard digging sand, gravel and clay, interspersed with large boulders.


Section E-E
Section F-F
Section D-D
Section H-H
Typical Concrete Joint
Sectional Plan
Longitudinal Section B-B
Longitudinal Sectional A-A
DETAILS OF
SCREEN HOUSE AND GRIT CHAMBER,
NEW BEDFORD
INTERCEPTING SEWER

The excavation was started with a scoop dredge at the outer end and carried towards the shore. As the work was nearing completion a ledge of solid rock was struck and it was necessary to remove 340 cu.yd. by blasting. The rock was under about 20 ft. of water and was drilled by a so called drill boat, a scow with two 3-in. diameter steam drills mounted on the forward end. After the rock was broken up it was removed partly by a scoop

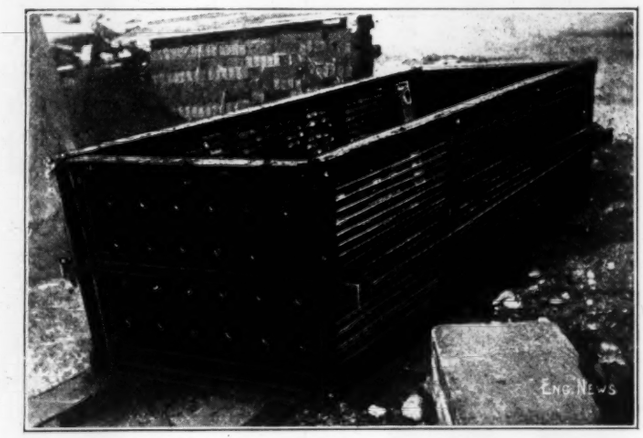


FIG. 5. VIEW OF BOX OR GAGE SEWAGE SCREEN, NEW BEDFORD, MASS.

dredge and partly by a lighter and diver. The excavation was not classified, all being included at 55c. per cu.yd., so that we have no prices for the rock excavation alone.

The laying of the pipe was from a lighter (Fig. 6), the pipe being assembled on shore in 48-ft. sections, weighing about 30 tons each.

The joint between the sections was calked under water by divers and bolted by two bolts, one on either side of the pipe. This joint is not the regular water-pipe joint but a special tapering spigot joint which could be made up on shore and then pulled apart so that it was only necessary to enter the pipes in position under water and hammer the lead tight.

The pipes have been tested every 400 ft. by placing a bulkhead at the end and submitting the line to a hydraulic pressure of 5 lb. per sq.in. A leakage of ¼-gal. per lin.ft. of pipe per hour was allowed. There has been no trouble in keeping well inside this limit of leakage.

The outfall was originally designed with a distributor outlet; that is, a large number of small holes along the pipes near the ends to diffuse the sewage through a large volume of water. This plan was finally abandoned as unnecessary and the present outlet for each line of pipe is through an upturned quarter-bend the full size of the pipe. The quarter bend is in two pieces for convenience in handling. It is believed that under so much water the sewage rising by its lower specific gravity and higher temperature will spread out into a wide conical shape and disperse in the currrents with reasonable rapidity. The outlet end is set in a large mass of concrete to insure stability.

GENERAL

All machinery at the pumping stations, gate chambers and screen house will be operated by electricity, which will be furnished from a power plant erected as a part of the screen house.

The sewage-disposal system is about one quarter completed. The total cost of the work wil be about $1,600,000. Of this the outfall will cost about $200,000, the screen house with grit chamber and power plant about $80,000, one pumping station with 10,000,000-gal. daily capacity about $23,000, and three other pumping stations about $5000 to $10,000 each.

The work now under way is in five sections, four let out to contract and one being built by city day labor.

Construction work was started on the submerged outfall section in January, 1912, and on the reinforced-concrete sewer in open cut in April, 1912. Up to Apr. 1, 1913, about $180,000 has been spent, about 30% of the total estimated cost of $1,600,000. This comprises completed work on one of the 60-in. lines of submerged outfall, a tide gate and sluice gate chamber, 10,200 lin. ft. of 7x7⅔ ft. reinforced-concrete sewer, 1800 lin.ft. of 4x4⅙-ft. reinforced-concrete sewer, a pumping station shown on the plan at Cove road, and some short tributary pipe sewers.

The total value of work already awarded is $600,000, including the screen house and about 3000 lin.ft. of 7x7⅔-ft. reinforced-concrete sewer. This latter work is

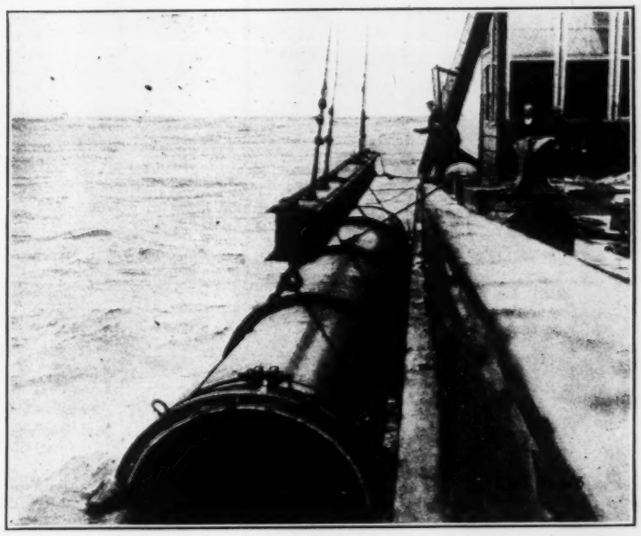


FIG. 6. LOWERING 48 FT. OF 5-FT. CAST-IRON PIPE FROM A LIGHTER, NEW BEDFORD OUTFALL SEWER

just being commenced. The lower half of the system, at the south end of the city, will be ready for operation about December, 1913, but the whole work will not be completed before April, 1915.

The system was designed and brought to its present stage by William F. Williams. Mr. Williams was formerly city engineer but was recently appointed chief engineer of the Massachusetts Harbor and Land Commission. He still continues to have supervision of the work as consulting engineer.

A Conference on Concrete Road Building is to be held in Chicago, Feb. 12-14, 1914, in connection with the Cement Products Exhibition. The temporary chairman of the conference will be Dr. W. F. M. Goss, of the University of Illinois. An advisory board made up of prominent engineers and others interested in road construction has been organized. The purpose of the conference is to bring together the latest experience in the construction of concrete road surfaces and foster better practice in this class of road construction.

The Latest Bridge Design for Sydney Harbor, N. S. W.; the Third Largest Span in the World

The subject of the crossing of the harbor between Sydney and North Sydney, New South Wales, seems to have as perennial an interest as the Hudson River Bridge at New York City. As early as 1878 there were projects for a fixed bridge and a floating bridge to connect the two portions of the growing antipodean city, and in 1901 the agitation crystallized in a government advertisement for competitive designs and bids for a bridge between Dawes Point and McMahon's Point in North Sydney. The board appointed to pass upon the designs of the bridge recommended the adoption of that design prepared by J. Stewart & Co., of Sydney. This design was illustrated in ENGINEERING NEWS, Sept. 22, 1904. It provided for a cantilever bridge having through trusses and a single deck with a channel span 1350 ft. long and anchor arms of 500 ft. at one end and 580 ft. at the other end. Long approaches were required in order to reach the elevation necessary to give a clear headway of 170 ft. above high water at the middle of the channel span. The bridge was to carry at one level a 35-ft. roadway, a double-track street railway and a double-track steam railway. There were also two 10-ft. sidewalks. The total estimated cost was $9,700,000, of which the main bridge, 3030 ft. long, represented $6,825,000. Several American firms were interested in the competition and it looked at one time as though the bridge would be built by American engineers.

On account of financial depression, nothing was done toward the construction of this bridge, but in May, 1908, a commission was appointed to investigate and report upon the best practical method of establishing direct communication between the northern and southern sides of the harbor, which communication would avoid both obstructions in navigation and provide the best route for traffic. The report of this commission was made under date of Mar. 29, 1909, and was reprinted in ENGINEERING NEWS, July 29, 1909, p. 128.

After reviewing the various kinds of bridges which have been proposed for the different crossings, and the different types of sub-aqueous tunnels which had developed up to that time, the commission reported in favor of a number of different tunnels connecting the two parts of the city. The bridge was rejected on account of its great height above the water and the consequent long approaches which would make the ferry trip above the shore line much shorter than the bridge trip, and, therefore, much more used by those requiring to cross the harbor. The report further stated that to increase the capacity of such a bridge in the future would be almost impracticable and certainly very expensive. On the other hand, additional tunnels could be built without difficulty if traffic might require. The different tunnels recommended were for street railway, steam railway and highway traffic, and were to be built by approximately the same method as was used in the well known Detroit Tunnel.

As in the previous case, however, this recommendation was allowed to lapse, and early in 1912 the congestion of traffic in Sydney and the very evident need for a crossing between the two portions of the city brought about action on the part of the government. David Hay, a London Traction Expert, was brought to Sydney to investigate the subject of transportation and J. J. C. Bradfield, of the Public Works Department of New South Wales, was appointed Engineer-in-Charge of the Sydney Harbor Bridge and City Transit. As a result of their investigations, a recommendation was made last winter and was adopted on May 30, 1913, by the New South Wales government, to connect Sydney and North Sydney by means of a bridge. This bridge was designed and the general scheme of its relation to the city railways worked out by Mr. Bradfield. The bridge has a somewhat different location than that proposed in 1903, running from Dawes Point to Milson's Point, on practically the same line as the proposed highway and street-railway tunnel of the 1909 project. The new bridge is described as follows in *The Surveyor and Municipal and County Engineer*, July 11, 1913:

> The main bridge in the design consists of nickel-steel cantilevers, supporting central girders, also constructed of nickel steel. The anchor or shore arms of the cantilevers and the harbor or cantilever arms are each 550 ft. long, while the central girder is 500 ft. long. The main piers supporting the cantilevers are placed 1600 ft. apart, center to center. The total length of the steelwork is 2700 ft. Provision is made between the girders for four lines of electric railway and a roadway 35 ft. wide. Outside the main girders on the eastern side is a motor roadway 17 ft. 6 in. wide, and on the western side a footway 15 ft. wide. A headway of 170 ft. above high water is provided for shipping under the bridge. The main piers and the anchor piers are to be constructed of masonry of bluestone concrete. The approaches consist of reinforced concrete or steel arch spans. On the southern side the roadway and footway reach the surface of the ground at the intersection of Princess St. and Argyle St., while at the northern side the roadway and footway reach the ground at about the intersection of Burton St. and Alfred St. opposite the Town Hall, North Sydney.

In connection with the bridge, there has been also recommended a system of subways and rapid-transit lines through the city, which will connect with the bridge and outlying districts. This system is to be submitted to the government for approval very soon. The bridge itself is estimated to cost about $13,000,000, and the street railways and subways, according to our authority, about $5,000,000, although this latter figure seems to be very low.

The main objection to the tunnel proposed in 1909 is, according to Messrs. Hay and Bradfield, that the grade would have been heavier and the distances by rail longer, under water, than over the water. The maximum rise against the train by bridge would be 136 ft., as compared with 181 ft. by subway. In his report, Mr. Hay estimated that the additional length and severer grade would add at least 15% to the cost of electric current alone, on one portion of the connection between the two parts of the town.

It is estimated that the bridge will be constructed and ready for traffic in seven years' time, although this figure is dependent upon the final adoption of the scheme by the government. At present, the scheme has been adopted by the Standing Public Works Committee of the New South Wales government, but the appropriation for the construction has yet to be made. In view of the previous history of the project, it is by no means certain that the construction will be carried out. If built, the Sydney bridge will be the third longest span in the world, being surpassed only by the Firth of Forth and Quebec cantilevers and being equaled by the Williamsburg (New York City) suspension span.

EDITORIALS

Some Reasons for the Increase in Public Debt and the Raising of Tax Rates

In connection with the perennial complaint of increased public indebtedness and growing tax burdens, it is generally overlooked that a good part of this tax burden really does not mean increased expense to the individual or to the property owner, but is due to a mere change from private to public control of industry. The bulk of the increase in public debt and taxation comes because the city, the state and the nation does for us all now what was formerly done by private enterprise, but none the less at our own charge.

By way of illustration: We now pay taxes for road and bridge construction and maintenance instead of tolls for the use of highways and bridges. We pay taxes for electric or gas street lighting instead of each householder buying a kerosene lamp and the oil to burn in it, and keeping it burning on a post in front of the house on dark nights. We pay taxes for municipal garbage and ash collection instead of hiring a private scavenger. These are but examples of a long list of supplies and services now furnished by some government body instead of by the private tradesman or the private corporation operating under a franchise.

Again, public debts and taxes are higher because the public demands vastly more than was deemed ample a generation ago. Perhaps this is extravagance in some respects; but in others it is public recognition of the fact that good public service, pure water supply, sanitary sewage and refuse disposal and well lit and well paved streets are worth what they cost.

It is little short of nonsense to call this change from public to private organization the growth of paternalism in government. The change is really a part of the great coöperative movement of the day. Of course, in the great shift from private to public coöperative service, a considerable increase in the burdens of taxation may fall upon men of wealth compared with those of small means. A part of this increase is justifiable beyond dispute as a contribution to the general welfare in accordance with ability to pay as well as in proportion to benefits received.

Where injustice prevails, it is chiefly because of failure to so adjust rates for certain services—like household charges for water and light—as to make them proportionate to cost and benefit. With a proper distribution of service costs between private consumers individually, and again between all private consumers on the one hand and the general public (as the city) on the other, the wealthy man pays only his due share of the cost of service, whether the service is private or coöperative.

There remains, it is true, the question of excessive cost under public ownership of public utilities as compared with private ownership; but we think a careful investigation would show that any excess of this kind which there may be makes up but a small percentage of the increasing burden of debt and taxation. Even should it be granted that water or light companies build and operate their works at less cost than do cities, it does not follow that the private consumer or the taxpayer gets the benefit.

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Fixing a Basis in Advance for Comparison of Alternate Bids

It happens very often in receiving bids for public work that bidders will be asked to quote prices on alternative methods or materials, leaving the decision to be made later as to which material or method shall be adopted. Desirable as this plan is, it often gives opportunity for charges that favoritism or even corruption has prevailed in the final award of bids.

We have before us a specification in which a simple plan was adopted to prevent the possibility of any such charges and enable the contract to be awarded to the lowest bidder under direct competition as laid down in the specification. The specification referred to is one in connection with the new supplementary water-supply for the City of Hartford, Conn., which is being constructed by the Board of Water Commissioners of Hartford under direction of Mr. Caleb Mills Saville as chief engineer.

A 42-in. pipe line about 7½ miles long is an important part of this work, and bids were invited both for steel pipe and for cast-iron pipe. To allow for the greater friction in riveted steel pipe, its diameter was fixed at 44 in. in comparison with 42 in. for the cast-iron pipe or for lock-bar steel pipe. In order to assign a definite value to the superior advantage of cast-iron pipe as regards durability, etc., the following clause was inserted in the invitation to bidders:

> In comparing the bids for the award of this contract, an addition of 15% will be made by the Board to the prices proposed for steel pipe. This addition is to be considered for the purposes of this award as the resultant difference in value to the Board between the steel and cast-iron pipe, taking into account durability, reliability, leakage and other pertinent factors.

The bids were canvassed on the basis above set forth and the contract was awarded on June 23 to the lowest bidder. It is noteworthy that while this low bid was for 42-in. cast-iron pipe at a total cost for the whole work of $427,538, the next lower bidder who offered 44-in. riveted steel pipe was only $1400 higher after the 15% per cent. addition was made to his bid, and the next bidder who offered 42-in. lock-bar pipe was only $3500 higher than the lowest bid. There would probably be a wide difference of opinion among engineers as to whether 15% was too much or too little to allow for the advantage of cast-iron pipe over steel pipe, but the canvass of the bids proves that the allowance was a surprisingly exact measure of the relative cost of these various kinds of pipe.

Railway Terminal Developments in Chicago

The railway terminal situation at Chicago has entered upon a rather unusual phase, since the city authorities have taken the position that the municipal aspects of the problem should receive greater consideration, in the interests of the city and the public. The city has decided, therefore, to have the several recent improvement projects investigated by an engineer on its own behalf, and to defer action upon them until his report is made. In another column we present some description of the several projects.

The recent developments in the terminal problem have been reviewed in ENGINEERING NEWS of Feb. 27 and June 12, but we may explain that the present situation has arisen from the proposition of the Pennsylvania Co. to rebuild the old Union Station on an enlarged scale and in line with modern ideas. This station is owned by the company and is used by five railways, two of which are parts of the Pennsylvania System. Four of these roads indorse the project, but the fifth seems to have taken no active interest in it. The plans have been in preparation for some years, and about a year ago the company was asked to make the general location conform to the "Chicago Plan" which had been drawn up by representatives of the city and various civic organizations, and which was largely the work of the late D. H. Burnham, one of Chicago's leading architects. This plan was prepared with a view to the systematic expansion and development of the city on both utilitarian (or commercial) and æsthetic lines.

The railway company, however, declined to consider this proposition, preferring to adhere to its present location. This resulted in some public opposition when its plans were submitted, and in the criticism of the plans several points of objection were raised. Following this, several projects were brought before the public and the municipal authorities providing for a general improvement of the terminal system as a whole, instead of the reconstruction of one station. Public hearings were held for the presentation and discussion of these projects, and the outcome was that the council committee on terminals appointed an engineer to examine the various plans and report upon the most desirable means of improving the terminal system, as noted elsewhere in this issue.

As suggested editorially in our issue of June 12, it is very desirable that instead of the city simply accepting plans prepared by the railways, there should be cooperation between the city and the railways in the preparation of the plans, so as to insure that the interests of the public and the municipality will be given due consideration. This is the position now taken by the Council Committee on Terminals, as noted above. In a recent letter, Mr. Delano, President of the Wabash R.R. (who has been interested in the terminal problem for some years) states that within the next few years large expenditures will have to be undertaken for the improvement of four of the six present stations, and the question for the public to consider is whether each group shall be allowed to work out its own solution (independently of other groups and of the city's street improvements and rapid-transit facilities), or whether the city shall insist that the railways coördinate their terminal facilities in order to provide for the interests of the public. He expressed the opinion that if this latter step should be taken by the city, the railways would sink their differences and agree to have a solution of the problem worked out by a committee of engineers.

For many years there has been a growing demand for the rectification of the Chicago railway terminals, and especially for the release of the large areas of land in the heart of the city which are owned by the railways and used mainly for freight yards and sheds. The arrangement of the freight terminal facilities is hopelessly confused and without any attempt at systematic planning or coördination. The facilities are quite inadequate for present requirements, and cause great congestion of street traffic. Further, the great area occupied forms an effective barrier to proper expansion of the commercial section of the city.

While the passenger terminals receive most of the public attention, it is the freight terminals which are the main source of the trouble. The passenger stations proper take up comparatively little space, and their railway approaches, where separated from the street grades, are not seriously objectionable. The principal objections to elevated approaches are the obstructions to main streets due to the bridges; these are usually low, of considerable width (making the street dark), and very frequently have lines of columns in the center of the street and on the curb lines. The freight yards, however, form an actual barrier, since the large areas occupied are crossed by only a few streets, the railways having been allowed to close many streets which should properly extend across their property. Much of the business done in these yards should be done in outlying yards and the railways are slowly making a move in this direction. The local business proper could be handled more efficiently on a much smaller area if planned systematically and on modern lines.

Various plans for improving both the passenger and the freight terminals have been put forward from time to time, and those for the passenger stations may be divided into three classes: (1) The improvement of one individual station, regardless of the terminal problem as a whole; (2) the establishment of a group of separate but adjacent and coördinated stations; (3) the establishment of one vast union station for the use of all the railways. The second and third plans provide for convenient transfer of passengers, baggage, etc., between different roads. This would eliminate the enormous amount of traffic of busses and transfer wagons which cross and recross the city in their routes between the present widely separated stations, thus adding materially to the traffic congestion on the streets. The second or multiple-unit plan has the advantage that the stations could be built and operated by groups of railways, while the third plan would require the coöperation of all the railways in forming a terminal organization to operate the one great station.

It is of interest to note that three of the present projects provide for loops of 180° in the stations to avoid all reverse movements. In two of these projects at least, the loops would be traversed by main-line trains and by every train using the terminal. Such an arrangement seems unlikely to commend itself to railway engineers or operating officers, owing to the possible difficulties of traversing such curves with long and heavy trains, and especi-

ally of starting such a train on these curves. The amount of rail wear and track disturbance would be very great, while the difficulties of adjustment and renewals would be serious. The loops in the Grand Central station at New York are for suburban trains and express cars only, and will be used only by a certain portion of the traffic. These loops have not been put in service and the same is true of the loops at the South Station in Boston. The latter were built several years ago and were intended for suburban service only.

Under the circumstances, the city is to be congratulated upon the way in which the authorities have handled the present situation, and Mr. Wallace's report may be expected to be of rather exceptional importance and interest.

The Buffalo Grain-Elevator Dust Explosion

Every engineering student and every engineer should have indelibly impressed upon his memory a clear understanding of the fact that any combustible material whatever, when in the form of fine dust, offers the possibility of a disastrous explosion. Every little while this possibility becomes a grim reality, as happened at Buffalo last month when a huge plant for drying and storing grain and manufacturing grain products was instantly wrecked with a loss of over thirty lives.

The Buffalo disaster, which was described on page 88 of our issue of July 10, is the most recent of such fatal calamities, and because of the large number of the victims and the wide publicity given to it, it ought to serve a useful purpose in impressing on the public as well as on engineers the inherent danger in all such combustible dust. Apparently nothing short of such serious and fatal disasters will impress the lesson of danger with sufficient force to induce precautions against such disasters.

Notwithstanding the many dust explosions on record, one reading a report of the testimony recently taken at the coroner's inquest in Buffalo on the victims of the Husted elevator explosion might suppose that such explosions had been practically unknown hitherto. For example, one of the principal witnesses at the inquest was the general superintendent of the mill, who is a graduate in chemical engineering from Cornell University. He described the various precautions taken to guard against fire and explosion, such as the wire-guarded electric-light bulbs, the placing of wires in conduits, the rules against smoking, the requirements that switching locomotives should not run into the buildings, but not a word as to the cause which in his opinion had produced the explosion. The assistant superintendent could give no information as to any causes which might result in such an explosion as that of June 24. The chemist at work in the plant's laboratory at the time of the explosion had no information concerning the dust in the mill and had not studied grain-dust explosions. A millwright employed in the plant testified that there was even less than the usual amount of dust on the floors as the place had been given a thorough cleaning a few days earlier. Another millwright gave similar testimony. Numerous other witnesses were brought forward to testify that the mill and its equipment were in good condition and apparently safe. Having developed by this remarkable testimony that there was no assignable cause for the explosion, the inquest was adjourned until Aug. 28, when experts will be called by the state in the endeavor to learn why the explosion occurred.

One can readily understand that in a coroner's inquest, where it is attempted to fix criminal responsibility upon some person or persons for a disaster involving large loss of life, there is a very natural tendency to cover up the facts, scientific and otherwise, and shroud the whole affair with a cloak of mystery.

To any competent engineer or chemist, however, there is no mystery whatever in what occurred at Buffalo. It is merely the last of a long, long series of dust explosions which during the past half century have cost thousands upon thousands of lives and many millions of dollars of valuable property. It is well established that in most coal-mining explosions the destructive agent is not gas, but fine dust. Flour mills, grain elevators, candy factories, starch works and numerous other industries in which fine organic dust is distributed through the air have each contributed its quota of victims to teach the lesson as to the explosibility of fine inflammable dust.

Mention may be made also of the numerous disasters which have occurred in connection with the burning of powdered coal. Many in the cement industry will recall especially the disastrous explosion at the Edison cement works nearly a dozen years ago in which Prof. Darling and a number of others lost their lives.

There is one point regarding dust explosions which deserves especial emphasis and which every engineer particularly ought to understand, and that is that only a very small quantity of dust is required to create a most violent explosion.

The proportions of dust to air for complete combustion may be set roughly as one pound of dust to from 10 to 20 lb. of air. By volume the proportion would be one part of dust to several thousand parts of air. But these proportions are not necessary to cause disaster. If the particles of dust floating in the air are close enough together so that flame may be transmitted from one particle to the next, there may be a fatal accident even if no explosion occurs. A writer in the *Railway Age Gazette* of July 18 describes a fatal disaster in the powdered-coal plant of a cement mill as follows:

> All these explosions are caused by impalpably fine dust floating in the air in suspension. This floats in layers or strata. At a recent explosion (?) in one of our biggest cement plants a foreman and some of his men were repairing and cleaning a coal-grinding mill. One of the men had rammed a piece of waste on the end of a stick into a part he was cleaning and somehow (no one knows) it caught fire as he pulled it out. Immediately there was a swift hissing sound like a pinwheel going off, or escaping steam, and in a flash this indescribable death traveled the length of the room, down a stairway and back several times in layers just like a train of powder, only there was no report, no explosion, just a hissing. The men came out—they were absolutely denuded, yet seemed to retain all their faculties. The foreman said: "I'm done for and am going to die." He still had enough life in him to tell what had happened before he became unconscious. They all died very shortly afterward.

With our present-day knowledge of the nature of combustion, there is no mystery whatever about what occurs in a dust explosion. Twenty-five or thirty years ago a popular scientific lecturer in England demonstrated in a most convincing way the nature of combustion and explosion. He had a series of chambers filled with various

combustibles of different fineness graded all the way from kindling wood down to fine shavings and then to fine dust. He thus demonstrated to the audience how the smaller the particles of the combustible, the more rapid is its combination with the oxygen of the air and the greater the resultant pressure. With fine shavings placed under a box, sufficient explosive pressure was produced as they were ignited to raise the box some distance; but whereas the shavings require several seconds to burn, fine particles of organic dust which can float in the air require only a very minute fraction of a second. When one clearly understands that even the explosion of such substances as gunpowder or dynamite is only an extremely rapid combustion, the mystery of the explosion of combustible dust floating in air ceases to be a mystery.

Safeguarding Life in Factory Fires

The fatal fire last week in a clothing factory at Binghamton, N. Y., in which some sixty persons—over half the number in the building—were burned to death, calls renewed attention to the pressing problem of safeguarding the lives of the millions of men and women who spend the daylight hours in crowded factories.

To those who have given careful study to this subject it is clear that many of the safeguards on which chief reliance has been placed in the past for the protection of life under such conditions are useless or even worse than useless. In the Binghamton holocaust, for instance, a part at least of the loss of life is claimed to have been due to the fact that as frequent fire drills had been held in the factory during the last two months, the girls employed in the factory thought the alarm sounded when the fire broke out was only a fire drill. Hence they were slow in responding to it and even stopped to get clothing, purses, etc., before going down stairs.

This terrible disaster at Binghamton makes especially timely the valuable paper of Mr. H. F. J. Porter, read at the recent Baltimore meeting of the American Society of Mechanical Engineers. Elsewhere in this issue we reprint Mr. Porter's paper practically in full; and we wish here to call especial attention to his recommendation, which appears to be by far the most practicable and meritorious suggestion ever made for safeguarding the lives of employees in the upper stories of factory buildings.

In the first place Mr. Porter demonstrates conclusively in his paper, what every engineer who has carefully studied the subject well knows; that by no form of vertical fire-escape ever built or proposed, whether it be stairways inside or outside, ladders, or any other means, is it possible for the hundreds of people employed on the upper stories of high loft buildings to reach the street in safety before a fire has spread fast enough to cause destruction. Mr. Porter also points out that making the building itself fireproof makes little difference in the danger to human life where industrial processes are carried on dealing with large quantities of inflammable material. This has been demonstrated by numerous fires which have occurred in buildings of fireproof and slow-burning construction. Mention may be particularly made of the fireproof Asch Building in New York City, in which over a hundred girls employed by the Triangle Waist Co. were burned to death two years ago. Reference may also be made to the terrible loss of life in the Iroquois Theater fire in Chicago at an earlier date in which the structure of the fireproof theater building itself was practically uninjured.

Mr. Porter's proposal for a manufacturing loft building in which the lives of the occupants would be effectively safeguarded is substantially as follows: Near the middle of every such building have a substantial fire wall, extending from the foundation to the topmost story and dividing the building into two parts. On each floor of the building this fire wall would be pierced by two or more doorways equipped with automatic fire doors. Each half of the building on either side of the fire wall would have stairways extending from the top of the building to the bottom. As this vertical fire wall through the building really converts the structure into two separate buildings, fire may start in one of these buildings and spread to a considerable extent before the other building will become endangered.

If fire breaks out on any floor of this building, employees on that floor can quickly escape through the doors in the fire wall to the other side and the fire doors closing after them prevent the flames following. If the fire spreads so that other floors are threatened, the employees there can likewise make their escape to the other side of the fire wall. Then if the building on that side of the wall should eventually be endangered by the burning of the other half of the structure, there would still be enough delay to permit all the occupants of the building to make their way down the stairways to the street at comparative leisure. The fact that they were in no immediate danger and that the flames were not following close after them would doubtless be sufficient to prevent a panic and resulting loss of life. Mr. Porter's plan is not applicable solely to loft buildings used for manufacturing purposes. In department stores, for example, which have long been realized to present the possibilities of a terrible holocaust, such a vertical fire-wall division separating a building into two parts transforms present dangerous conditions into comparative safety. Hospitals and asylums are another class of buildings in which this offers practically the only possible means of reasonable safety for the inmates.

We recognize, of course, that to carry out Mr. Porter's suggestion in the best possible manner the building should be constructed in this manner originally, and, furthermore, the building should cover a fair-size area. Many property owners controlling only a small plot of land naturally desire to build upon it a many-story structure too narrow to permit of its economical division by a fire wall through the center.

Such considerations, however, ought not to stand in the way of securing public safety. It would be difficult to exaggerate the amount of class feeling and hatred of labor for capital which has been stirred up by the loss of life in the Triangle factory fire. It is well known that labor agitators and socialistic and anarchistic orators make use of every such occurrence to incite enmity against capital and property interests.

This fact alone makes it eminently worth while to enact stringent legislation for the reasonable protection of the lives of factory workers. A considerable number of laws and factory ordinances were put in force as a result of the Asch Building fire; and already it is announced that state labor departments and state legisla-

tures are at work framing new ordinances as a result of the holocaust at Binghamton.

The trouble is that all the safeguards on which energy has been hitherto expended are fatally ineffective. Every engineer who follows Mr. Porter's careful analysis elsewhere in this issue, must admit that his demonstration is conclusive as to the limited capacity of all means of vertical escape from a crowded high building. The fire-wall division, however, gives to the employees on the upper floors of a building nearly as ready means of escape as is available to those working in a one-story structure. The plan deserves, therefore, the widest possible publicity. We believe a large part of the present stringent measures in factory legislation and labor-department supervision over workshops aimed at safeguarding from fire and panic could much better be expended in promoting the fire-wall method of protecting life.

As we have said before, to carry this suggestion out in the best possible manner the building should be constructed originally with such a fire wall and with stairways at opposite ends of the building. There are, however, many buildings already erected, especially those of fireproof, or slow-burning construction, which could be at comparatively small expense remodeled according to this plan and which if so altered would be vastly safer for their occupants than they could possibly be made in any other way.

It may be remarked in conclusion that when Mr. Porter's paper was read at the Baltimore meeting of the Society in May, it received the highest commendations from those present. It is to be hoped that engineers, who can best appreciate the value of Mr. Porter's suggestion, will take every possible means to give it such further publicity as may be in their power.

LETTERS TO THE EDITOR

A Comparison of Old and New Designs for Vertical-Lift Bridges

Sir—In the description of direct-lift drawbridges without cables in ENGINEERING NEWS of June 5, it is stated that "up to the present time all these bridges have been operated by cables and winding drums." This was corrected in your issue of July 3, where it was shown that a lift bridge at Dresden was balanced by levers and worked by toothed segments without the use of chains, cables or sheaves. This bridge was quite a small affair 12 ft. long, 29.5 ft. wid, with a lift of only 5.2 ft.

The article is further incorrect in reference to the method of operating lifting spans. They were not all "operated by cables and winding drums," for several of the early ones were worked by water ballast. A design made in 1872 by E. T. Laing for a lift bridge at Newport, England, was intended to be operated by admitting and withdrawing water from the counterweight boxes in the towers. A similar method was put into execution in 1873 at Spencer Dock, Dublin, and in 1878 on a bridge at Calcutta, India. During the last ten years, new lift bridges over the Erie Canal in New York State have been operated by pistons under hydraulic pressure from the city mains. An example of these is a bridge at Lockport, completed about 1900.

Lift bridges have been operated frequently by rack and pinion gearing, the racks being fastened to the tower posts, one of the first thus equipped being that of 1848 over the Grand Surrey Canal (England). Racks and pinions were also used in 1868 on a lift bridge at Paris, and a design for the New Bremen bridge, made by the writer in 1904, showed a similar arrangement (*Electrical Review*, Dec. 31, 1904). In this case the gearing and motors were placed beneath the floor, as shown in your issue of June 5, for a bridge at Pine Bluff, Ark.

The statement that all these bridges "have been balanced by counterweights suspended from other cables," is also incorrect. During the last ten years, air-tight pontoons or buoys have been extensively used as substitutes for counterweights, examples being those in the lift bridges at Lauenenburg, Germany and Rochester, N. Y., both completed about 1902. Floats were also used by Vescovali, in 1893, on a bascule bridge over the Tiber River near Rome. The placing of counterweights below ground is not a new departure on lift bridges, as might be inferred from your description, for they were so arranged by J. J. Hood in 1848, and were similarly disposed on a bridge at Paris in 1886.

Direct-lift bridges with double balance levers, as shown in your article of June 5, are a modification of the familiar type known as the Dutch portal bridge, with the span rising horizontally instead of revolving about a shore hinge. A lever arrangement working on a fulcrum tower was installed at Dublin in 1873, the bridge being counterweighted by water ballast; the amount of water was increased or diminished for the purpose of raising or lowering the span, the movement being automatic.

Your article further stated that "there are objections against the cable system of operation, such as the possibility of breakage, cost and trouble of renewing the cables, adjustment for stretching, and the general care of the cables and sheaves." These objections are common to structures and machines of nearly every kind where parts wear out and must be renewed, and the disadvantage of cables in lift bridges is much a matter of opinion. It is true that several unfortunate accidents have happened, due to the breakage of cables, but these have been no more frequent than the accidents to other kinds of moving bridges, and it is well known that some of the foremost bridge engineers specify cables or chains, rather than more rigid forms. Chains were used on a lift bridge in England more than 60 years ago; they were preferred also in a bridge at Paris in 1868, and later on its renewal. A recent lift-bridge design patented by Frank C. Osborn includes eye-bar cables passing over hexagonal sheaves at the tower tops, a most ingenious arrangement.

The method of supporting trusses from intermediate upper panels is not a new departure, because many de-

signs for bascule bridges with cables and shore towers have the leaves supported in this way. Direct-lift bridges with double-balance levers are practicable for small lifts only, the one shown in your description having a maximum vertical movement of 32 ft. It is difficult to see how a rise of 150 ft. could be economical in this type of bridge.

In reviewing the general proportions of direct-lift bridges which have been built, it appears that in only one case (the cable-operated bridge at Kansas City) has the span length exceeded 300 ft., as proposed by Oscar Roper in 1867. The Hawthorn Ave. cable-operated bridge at Portland, Ore., has a moving span of 245 ft., about the same as the lever-operated bridge proposed for Lake St., Chicago. Few designs for direct-lift bridges have excelled that made in 1850 for crossing the Rhine at Cologne, where the estimated cost of the whole structure was $1,184,000.

Much emphasis is often placed on the adaptability of lift bridges to shifting river channels, a condition which has occurred on the Missouri and other rivers in the prairie states. It is interesting in this connection to note that a design of this kind with a succession of cantilever spans appeared in Europe in 1890. The bridge was a deck structure and towers for the lifting span could be moved to suit the position of the deepest water.

The use of stairs to reach the span when raised or partly raised, is common practice in Europe and has several times been proposed in America, as on one of the Newton Creek designs of 1896. The revival of old drawbridge inventions was referred to at some length by the writer in ENGINEERING NEWS, Feb. 15, 1912, and to those who are familiar with the history of bridge engineering it would seem difficult to present any device that is wholly new, or without some precedent.

H. G. TYRRELL.

Evanston, Ill., July 12, 1913.

The Limitations of Artificial Stream Controls for Gaging Stations

Sir—I have read with interest the article by C. Robert Adams in your issue of June 26, 1913, upon the establishment of artificial controls for stream-gaging stations, and although there is no doubt but what there is decided merit in such a control for certain streams in most localities, it falls far short in many others. This is especially true of a flashy stream that carries much light sand and gravel. The writer a few years ago had occasion to make very extensive measurements over a very carefully made concrete apron that extended for a distance of about twenty feet up and was downstream and was raised slightly above the bed of the stream. This apron extended the entire width of the stream.

The results were very discouraging and instead of being able to establish a true rating curve with a minimum number of gagings, it was necessary for the gager to practically live at the station in time of high water and gage continually, as the sand settled over the weir apron to a depth of eight feet, varying according to the stage of the flood. This sedimentation would be in evidence for several days following the storm but would eventually disappear, leaving the stream bed swept clean, with only the large rocks along the stream bed remaining.

My conclusion is, therefore, that in some places the control is extremely valuable and in others it is not worth the cost of placing. In other words, the artificial control is not a panacea for all stream-bed irregularities at a gaging station.

G. S. STROUT.

712 Hearst Bldg., San Francisco, Calif.
July 11, 1913.

More "Notes on the Pitot Tube"

Sir—This is to amplify the remarks I made in my article, "Notes on the Pitot Tube," published Apr. 17, 1913, in ENGINEERING NEWS and to comment on some of the fallacies which Mr. Guy introduces in his article "Origin and Theory of the Pitot Tube," published June 5th, 1913, in ENGINEERING NEWS. As Mr. Guy devotes considerable length to discussion of my article, I will answer his points in order.

To have experimented with moving water would have been very unsatisfactory in my case because the most important thing, the speed of the water, would depend on some questionable measurement. As for the effect of a pulsating current, I did not, nor did I intend to, discuss at all in my article. To have experimented with a fluctuating current would have been a complete waste of time unless the nature of the pulsation had been accurately known.

Mr. Guy does not devote much attention to my vulnerable point (if any), viz., the assumption that I made, so a more definite statement of my position may be advantageous at this point. First, I tried three different diameters of plain piping and found that $h = \frac{v^2}{2g}$. I am aware of no theory which explains this; to say that $h = \frac{1}{2} \cdot \frac{v^2}{g}$, where ½ is empirical, seems decidedly irrational. Pitot's axiomatic statement that $h = \frac{v^2}{2g}$ is not a proof, and was evidently not accepted as true, judging from the prominence given to the $h = C\frac{v^2}{g}$ formula. I next developed a legitimate proof which, if rigid, would apply equally well to all forms of mouthpieces. This I doubted, thinking that eddies would be set up in the case of the flat plate, and possibly of the conical mouthpiece, thus invalidating my assumption. My doubts were unfounded.

This assumption is that at all points in front of the opening there is either stream-line motion or total quiescence. I believe that this becomes slightly untrue as the speed increases (see article). The quiescent water must merge into the other gradually, or eddies are generated. There is no *proof* that this water is quiescent; but there is the following evidence suggesting that as the logical starting point for a theory:

(a) Observed phenomena of a ship as mentioned in my article.

(b) The experiments of Dr. Ahlborn "Die Wiederstandsvorgänge im Wasser an Platten und Schiffskörpern," published in "Jahrbuch des Schiffbautechnischen Gesellschaft," 1909. In these experiments Dr. Ahlborn towed various objects through water and photographed the motion of the water. He accomplished this by put-

ting fine oak sawdust in the water and sending a strong beam of light through at the time of photographing. The great number of interesting photographs in this paper show very clearly the smooth motion in front of a body, but the strong eddies at the back.

(c) *If the cone of dead water were turbulent, is it rational to suppose that such consistent results would be obtained? Surely the degree of turbulence would be affected by the shape of the mouthpiece. The results I obtained with different mouthpieces appear much too consistent for the vagaries of eddies.*

As the speed increases, I believe that the quiescent water and the stream lines become divided by a *layer* of rollers, or very small eddies, thus permitting the bounding stream line to have a small finite velocity and, consequently smaller pressure, which again necessitates a smaller pressure throughout the dead water, and this in turn supports a column smaller than $\frac{v^2}{2g}$. The maximum effect of this is slight for ordinary velocities. In my experiments this probably accounts for the fact that the results were always slightly below $\frac{v^2}{2g}$; whereas, experimental error should have played on both sides equally.

The impact theory, as originally suggested, gave $h = \frac{v^2}{g}$. Now clearly, the water is not turned through 90°, so Weisbach suggested $h = (1 - \cos \alpha) \frac{v^2}{2g}$ as being nearer truth. This is not as erroneous as the former, but of what use is it, α being an unknown quantity! Moreover α will be different for each infinitesimal annulus of water (assuming a circular obstruction). Mr. Guy blandly takes $\alpha = 60°$, without giving any reason, other than that he wishes to arrive at the result $h = \frac{v^2}{2g}$ As any result lying between the limits $h = 0$ and $h = \frac{v^2}{g}$, could be deduced by the same reasoning, it is fallacious to attach any importance to such an elastic result.

At the end of paragraph 2 is the following: "The 120° angle is certainly not out of reason, and after going so far we might as well agree that the generatrix of the cone could be sinusoid, in which case, the impact formula would still apply in a general manner; the main point being to determine the exact shape of the generatrix." Exactly! that *is* the main point and that is why it is useless to seek a simple proof along these lines.

In paragraph 3, Mr. Guy again discusses the contour of the wedge. I do not think this at all essential; the proof I suggested is quite independent of the shape of the dead water. The suggestion of a paraboloid is, however, probably very far from the truth, there not being a close analogy between an object moving through incompressible water at a low speed, and an object moving through highly compressible air at an extremely great speed.

The last part of paragraph 3 is surely unnecessarily confusing. In discussing stream-line motion one does not need to consider every individual stream line and then, in order to get the pressure at one point, approach that point by a path normal to the stream lines integrating pressures *en route*.

In Mr. Guy's discussion, following the quotation from my article, he takes a metal cap and discusses the situation thus created. There is a difference, however, as both the cap and the water are not solid, as is stated. The pressure at all points of the cap need not be the same; but, at all points of the water cap, it *must* by hydrostatics. If a cap could be shaped in exactly the form of the water cap (form unknown), then a height $\frac{v^2}{2g}$ would be maintained; but for any other shape, say a cone as suggested, some of the stream lines would touch the cone with part kinetic head and consequently diminution in the static head and, therefore, less end thrust on the cap.

I should appreciate a reference for one or more of the experiments which "totally disprove that the outer surface of the dead water is not a region of high pressure." If this be true, what prevents the water from running back from the tube? Mr. Guy appears to have difficulty in finding room for sufficient stream lines for the velocity to vary from zero to the normal velocity of the stream without having a discontinuity. It is to be remembered that the width of a stream line is limited only by the mean distance apart of adjacent molecules, and that, if necessary, the velocity could vary from zero to 50 ft. per sec. in a ⅛-in. space, without breaking the condition that adjacent stream lines vary by more than an infinitesimal. Mr. Guy's trouble appears to lie in imagining stream lines to be of considerable breadth.

As to the last part of paragraph 5, the experiments have nothing to do with my discussion as they deal with rapidly fluctuating velocities, however, as this topic has been introduced, the time is opportune for discussion. Firstly, I would like to point out that it was scarcely a wise thing for Mr. Guy to refute statements about occurrences in water by reference to experiments in air—a highly compressible substance. Secondly, if Mr. Guy's quotation were true there would be a decided paradox. He states: "But when the tube was made to oscillate by an ingenious pendulum device, such that the tube remained in one current as long as the other, the time of oscillation being adjustable at will, it was found that the reading of the tube was greater than would be produced in the high-pressure, or high-velocity current." What actually occurred was that the reading of the tube was the arithmetic mean of the two separate readings which was to be expected.

A column of water was used to measure the air pressure. If the pressure of the air fluctuates between two values too rapidly for the column of water to respond, if the oscillation of velocity be sufficiently rapid, the column of water will settle down to a steady value. To take a concrete case in which the ratio of the two velocities is 2:1; the two corresponding heads will be as $2^2 : 1^2$, or 4:1. The head actually registered will be $\frac{4+1}{2} = 2.50$. The average velocity is 1.5; squaring this gives 2.25 instead of 2.50 as found, thus showing an apparent magnification in head of $\frac{2.5}{2.25} = 1.111$. But this disproves the fundamental truth of $h = \frac{v^2}{2g}$ no more than an alternating-current meter disproves Ohm's Law because it gives the square root of the mean squares in-

stead of the simple average of the variable current. The parallelism of the two cases is obvious.

It is worth while investigating the effect of a rapidly fluctuating current, pulsating more as it would under practical conditions. Prof. Rateau experimented with a current that changed instantaneously from one extreme value to the other. This was done, I suppose, on account of practical convenience. It is clear, however, that this could not occur in practice but the velocity would vary gradually from one extreme to the other. If the velocity be plotted as a function of time the most rational guess would be a *sine* curve. Let velocity expressed as a function of time be $V = V_o\,(1 + k\,sin\,qt)$; here V_o = mean velocity, $\frac{1+k}{1-k}$ = ratio of extreme velocities. Following the same law as in the experiments, the head would be given by $h = \frac{1}{T}\int_o^T \frac{V_o^2\,(1 + k\,sin\,qt)^2}{2g}\,dt$, where T is the time of a complete cycle. The result comes out $h = V_o^2\left(1 + \frac{k^2}{2}\right)$. If the ratio of the extreme velocities is 2:1 then $k = \frac{1}{3}$ and $h = 1.055\,\frac{V_o^2}{2g}$. The limiting value when velocity varies between $2V_o$ and zero is $h = V_o^2\left(1 + \frac{1^2}{2}\right) = \frac{3}{2}\,V_o^2 = 1.5\frac{V_o^2}{2g}$.

A ratio of extreme velocities of 2:1 seems to me to be excessive and much greater than is likely to be met with in any case where a Pitot tube would be used. In this case, however, the coefficient would be only 1.055 in the equation $h = C\frac{V^2}{2g}$. It must be remembered that this matter remains to be tried out in water. Prof. Rateau thinks that the results would be the same as in air but experimental corroboration is desirable.

JOHN AIREY.

Engineering Department, University of Michigan, Ann Arbor, Mich., June 27, 1913.

NOTES AND QUERIES

The question whether a City Engineering Department should do city surveying for private parties in competition with private engineers, is raised by a correspondent in a Southern city. He writes:

The Engineering Department of this city is doing all sorts of private work, in direct competition with private engineers. In fact it has a monopoly, for the Building Inspector is not permitted to issue a permit to build until the owner can show a receipt from the City Engineer for the survey of the street lines of the lot. The City Ordinance covering the matter reads as follows:

When the services of the City Engineer are needed by any citizen to make any survey, or define the lines of lot, or to give levels, grades, diagrams, etc., application for such services shall be made at the office of the Board and entered in a book kept for the purpose. And the Board shall establish reasonable rates of compensation for such services, to be collected by the City Engineer, in advance from the party for whose benefit the services may be rendered, and covered monthly into the City treasury.

The department seems to construe this law so that its employees are permitted to do private work in the suburbs of the city, in some case many miles outside of the city limits.

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The Shuman-Boys Sun-Power Plant, near Cairo, Egypt, is to be tested and reported on by A. S. E. Ackermann, who recently left England for that purpose.

The Life Hazard in Crowded Buildings Due to Inadequate Exits*

BY H. J. F. PORTER†

Buildings in general are either non-fireproof or fireproof. The former can be compared to a pile of kindling wood cut in the open, sometimes oil soaked and always ready to be set on fire. The latter are comparable to a stove full of fuel ready to be set on fire. In both cases the human occupants swarm around in the interstices in the pile of fuel, and as soon as the fire starts those caught in the fagots have to work their way down through the smoke and flames to the ground to save their lives.

Factory buildings in particular are sources of great danger to their large number of occupants, both on account of their non-fireproof construction and because of the obstructions to rapid egress, due to haphazard placing of machinery, furniture and partitions and the small number, size and character of the exit facilities.

Of late, there has been advocated the unrestricted use of fireproof construction in the buildings themselves and the author has recommended the development of a form of exit drill of the occupants of each building to determine if, in the case of danger, they could escape readily from the building and if they could not, the alteration of the exits until they could. By "readily" is meant within three minutes, for from many conferences it was found that people do not want, nor would it be safe, to remain in a burning building longer than that time.

The capacity of a stairway, if time is not a factor and a stream of people pours into it only at the top and out of it from the bottom, is unlimited; but if time is to be considered the capacity is limited by its cross-sectional area. In a multi-storied building with crowds of people on each floor trying at different points in its length to get on to one stairway in a limited time, the conditions are very different. If more people try to get on to the stairs from each floor than the section between that floor and the floor below will hold, a jam will occur so that the flow downward will cease. The capacity of this section is very limited.

The capacity of a stairway of the average height of from 10 to 12 ft. between floors and not less than 22 in. wide would be one person on every other step or 10 and 12 per floor respectively, and if the width is doubled (not less than 44 in.) so that two people can come down abreast, twice those numbers or 20 and 24. If a stairway has winders in it, its capacity is reduced 50%. One person can descend a single flight of such steps 10 to 12 ft. high in 10 seconds, striking a gait which he can maintain for seven or eight flights of steps. After that he goes slower, making the tenth flight in about 11 or 12 seconds. Every person added in single file adds 1 second to this time. A double file takes no longer if the stairs are double width. Thus it will take 10 seconds for 10 or 20 people, that is, the full capacity of a flight of steps, to come down one story. The capacity of a stairway may be thus increased by widening it in multiples of 22 in. A crowd of people cannot be depended upon to come down more than ten stories. One or more of them will give out, and demand the attention of others. Those who do get down will be severely taxed. The total time required to empty a building is determined by the time required to empty either the floor farthest from the ground or the floor occupied by the greatest number of people.

FORMULA FOR EMPTYING A FLOOR BY ONE STAIRWAY

Number of couples (number of people divided by 2)	c
Time of formation in line after signal, seconds	10
Time one couple takes to march to top of stairs, seconds	10
Time each couple takes to pass through door at top of stairs, seconds	1
Number of stair flights (one less than number of floors)	f
Time of one couple to descend one flight of stairs, seconds	10
Time of one couple to go from foot of stairs to street, seconds	10

Total time = T = 30 + c1 + f10

Example. Time of emptying 100 people from tenth floor.
T = 30 + 50 + 90 = 170 seconds = 2 minutes, 50 seconds

Example. Time of emptying a ten-story building with 20 people on each floor is the same as emptying 20 people from tenth floor.

T = 30 + 10 + 90 = 130 = 2 minutes, 10 seconds.

Tests of the capacity of fire escapes in a limited time gave the following results: A straight ladder, 2 per floor; ladder set at 50° to 60° with the horizontal, requiring people to go down backwards, 3 to 4 per floor; stairs 30 in. wide, 10

*A paper read at the Baltimore meeting of the American Society of Mechanical Engineers.

†Consulting Engineer, 25 W. 39th St., New York City.

to 12 per floor; and the modern outside stairway with a mezzanine platform 40 in. wide, 20 to 24 per floor, the same as an inside stairway. Fire escapes are usually so exposed to flames from windows opening upon them that they are more often fire traps than fire escapes. They should be prohibited by law and safer methods of escape provided.

In order to insure the safety of the occupants of a building in case of emergency, one of two things has to be done: (a) there should be two stairways so that if one is cut off by flames or smoke the other can be used and the number of occupants reduced on each floor to meet the limited capacity of the part of the stairway between floors, or (b) the number of stairways increased so as to have two separate and independent stairways from each floor to the ground with its own exit from the building. People can then pour into the top of whichever one is not cut off by the fire and continue down and out at the bottom without colliding with those from any other floor. Fire drills installed under either of these conditions worked more or less satisfactorily, and the author tried unsuccessfully for years to have ordinances passed in New York City and legislation enacted at Albany, making them mandatory, but the expense of changes in the buildings and the idea of having employees walk out of a factory while manufacturing operations were under way, upon the sounding of an unexpected signal, did not appeal to factory proprietors as practical. It required holocausts in New Jersey, Pennsylvania and New York finally to bring about the legislation in those states.

As time passed, however, the author developed what might be termed an exit test in factories which presented the opportunity and found to his astonishment that almost without exception, exit facilities adequate for handling the regular number of occupants under emergency conditions, were lacking.

This situation has probably developed with the rapid growth of industry where a factory building had been built to accommodate a certain number of people, and then, as the business grew, more people were accommodated without realizing that each additional person became an increment of danger to all. Or, if the danger was at all appreciated, some means of escape from windows was supplied which might be anything from a rope to a ladder. After this condition had become general it crystallized into custom, and new buildings with exit facilities inadequate for their occupancy were designed, erected and accepted as safe. Ropes were followed by ladders, and these in turn by fire escapes which became in time an established necessity.

Engineers, when called upon to supply a mechanism, are expected to have it subjected to a working test, which it must pass before they get paid for it; but architects and builders have never been called upon to demonstrate by actual test that the facilities which they have supplied in their buildings for the purpose of emptying them under emergency conditions will actually work, and this notwithstanding repeated instances of panic congestion on stairs, of people being burned to death on fire escapes, of elevators sticking from the warping of their runways from heat, etc.

When subjected to test, these exit facilities in many buildings have been found to be entirely wanting in adequacy, and when this fact was brought to the attention of those who were responsible, it has been surprising to find how readily they accepted the criticism. On the other hand, those who possess these unemptiable buildings are skeptical of such statements and unwilling to be persuaded that the buildings are not safe. They point to all the other buildings erected by reputable architects and builders and naturally are incredulous.

In order to empty these buildings, additional stairways had to be built and fire drills developed to take the people out. Such changes in the building are expensive, for two stairways have to be installed from each floor to the ground, so that if one is cut off by a fire, the other can be used. In many-storied buildings the number of stairways required becomes impractical. In addition fire drills are expensive to operate, for they involve not only loss of time of operatives and a break in the continuity of the process of manufacture, but the actual going down stairs and return of people, some of whom may be lame, others affected by weak hearts or lungs, others anaemic or originally weak, reduce the efficiency of the working force for a very appreciable time. If the drill takes place at the end of the day this criticism might be modified slightly.

Such is the situation in the usual type of factory building to be found in the average town where ground is cheap, buildings large and stairways broad. Turning now to the loft building used for factory purposes, the conditions as regards emptiability are found to be very much worse and have to be corrected in a different manner.

Let us consider for the moment a one-story or ground-floor factory building with a doorway at each side, one of which is cut off by a fire. The people can march out horizontally through the other doorway and nothing will impede this horizontal exit except the size of the doorway. If this is 22 in. wide, a single file of people can pass out in an orderly manner at the rate of one person every second. If it is 44 in. wide, a line of people two abreast can pass out in the same time. One hundred people can make their exit through one 44-in. door, therefore, in 50 seconds, or say one minute.

Now put another factory on top of this one with one hundred people in it. The doorway at each side will have to open on stairways which lead down to the doorways constituting the exits from the factory below. Suppose a fire occurs on the floor below, cutting off one of these exits, the 100 people on the lower floor immediately proceed to make their horizontal exit, while those on the upper floor proceed to make a vertical downward exit to reach the doorway out of which those below are moving. The result is of course a collision, the stream of people from upstairs coming down upon the stream of people on the ground floor on their way out. This collision prevents both the up-stairs stream from coming down and the down-stairs stream from going out. There is a complete lock, and the building does not empty.

Not only have we put one factory on another in the case of our loft building, but we have piled factory on

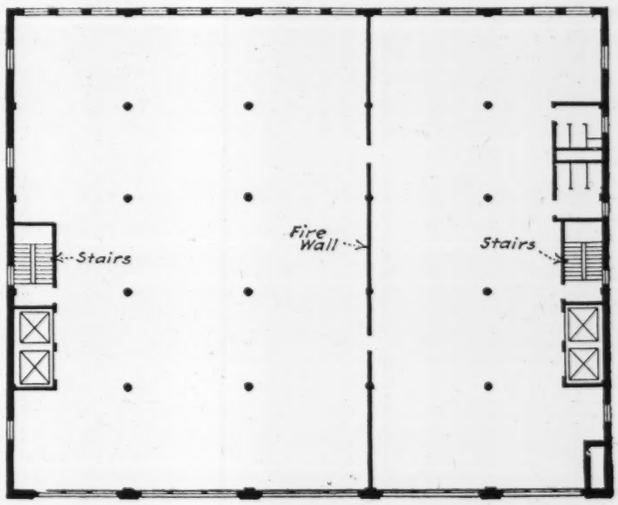

FIG. 1. FLOOR PLAN OF TYPICAL LOFT BUILDING, SHOWING FIRE WALL WITH DOORWAYS

factory until we have from 10 to 30 and more, one on top of the other; and each employing from 100 to 300 or more people. In cases of emergency as in the Asch Building fire, there are only two courses for the occupants: One is to burn to death, and the other to jump to death—"to burn up or jump down."

It is impossible to reduce the number of people per floor to the capacity of the stairs, say 24 per floor. Even if that number would be all that a business required, in case of emergency they would have to go down stairs, and it is a physical impossibility for people to stand the exertion of a trip down more than ten stories without resting; and when they stop to rest they block the stream and obstruct its exit. Under these circumstances it is necessary to develop some other method for people in high buildings to secure safety. The following suggestion is offered to meet the situation.

It has been seen that a horizontal escape by people on the ground floor is readily secured. Let us see if a horizontal escape to safety for people at any height from the ground can be developed. Suppose a fireproof wall is built across the building from cellar to roof practically bisecting it in a way so as to have a stairway and elevator on each side. This wall should have at least two doorways in it at a considerable distance from each other and closed by self-closing fireproof doors (Fig. 1).

It is improbable that a fire will occur on both sides of this wall simultaneously. It could occur only by incendiary origin, and that would hardly be possible in working hours. Should one occur on either side, the people on that side would go through the doorways in the fire wall, close the doors after them and be perfectly safe. That half of the building

in which the fire might be should be emptied in less than a minute if there were no more than 100 people on each floor to pass through one doorway 44 in. wide. If the principle of the horizontal escape presented by the fire wall is included in the design of new buildings, a most satisfactory method of securing safety at comparatively small expense will be offered.

In every way possible the horizontal escape should be developed in old buildings and the vertical escape subordinated. Factory buildings adjoining one another may have doorways through their sides connecting them on various floors closed by fireproof self-closing doors, or may be connected by outside balconies built around the party walls; or, if of different heights, doors in the sides of one may lead out on the roofs of the others.

The fire wall bisecting the building as described makes practically two buildings, each provided with elevators and stairways. A fire on one side of the wall would be confined to half the building, and therefore the property loss would be reduced one-half. Only one-half the people would be endangered and have to move, and the distance they would have to go would be only one-half what it would be if they were on the ground floor of a building without a fire wall. They could remain on the same floor till the fire was extinguished, or could go down to the ground by the elevators operating under normal conditions.

The fire wall eliminates the necessity for a fire drill with

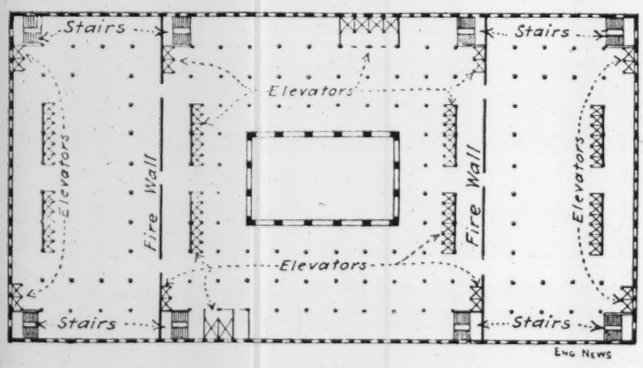

FIG. 2. SUGGESTED ARRANGEMENT OF FIRE WALLS, ELEVATORS AND STAIRS FOR DEPARTMENT STORE

its accompanying objections. Of course all buildings occupied by many people should have a fire-alarm signal system in them to advise the people promptly of their danger. In buildings where there is a fire wall the signals should be arranged so that in case a fire should occur on one side of the fire wall on any floor, a bell on each floor on the same side of the fire wall would ring, indicating on which floor the fire is. Then all the people on that floor and above it should pass through the fire wall and close the doors. Those below need not disturb themselves until the fire threatens them, and then they too can pass through the fire wall.

Each floor of our typical loft buildings is say 100x100x10 ft. and therefore contains 100,000 cu.ft. of air. The laws of New York and many other states require 250 cu.ft. of air per person as a limitation of occupancy. This limits the number of people per floor in a building of this size to 400 and if the stairways were 44 in. wide (and there are none now over 36 in.) at most only 40 per floor could possibly go down them even if the other 360 would let them.

With the fire wall, only 200 of the 400 people on each floor would have to move, and if there were two doorways in the fire wall at some distance from each other, they could reach safety through them in one minute, or if one were cut off by the fire, all could pass through the other easily in two minutes. More doorways can be introduced, and thus the time of exit could be lowered still further.

An effort is being made to increase the amount of air space required per person from 250 to 500 cu.ft., which would reduce the number of people per floor to 200, of whom only 100 would have to move, and they could easily reach safety in one minute.

The stairways and elevators should be inclosed in fireproof walls to prevent a fire on one floor continuing upward and setting the other floors on fire. The ceiling of the basement where the machinery is located should be fireproof, and should not be pierced inside of the building, so that a fire there would not reach the elevator shafts.

Fire escapes which are simply stairs and possess dangerous features not only of limitations as to size, but of accessibility for flames and smoke, should be looked upon as evidence of the incompetence or ignorance, or worse, of the architect, builder, or owner, and prohibited by law under a heavy fine. They are not only dangerous to life by giving a false confidence in their adequacy for escape, but they destroy the appearance of the building. Our cities should be built without such architectural blemishes.

Fire escapes of the chute type are tubes with a smooth helix instead of steps. If the only opening is at the top they have considerable capacity. They soon rust, however, and at best are not to be considered seriously in comparison with other means of safe exit. People cannot enter them at different floors while a stream of people is passing down.

The smoke-proof tower, claimed to have originated in Philadelphia, is the latest improvement in the line of fire escapes. It is simply an inclosed stairway on the outside of a building, but cannot be reached except by going out of doors. Its special claim is that smoke and flames cannot get into it. It has, however, no more capacity than any other stairway, and as its approach is always open to the weather and its interior is always more or less dark, it is never used in ordinary service and becomes neglected. These monuments to architectural incompetency can be seen here and there filled with the dust and accumulated rubbish of every unused open space. When a time arrives for using them, everybody has forgotten their existence. During the last year or two, notwithstanding the protests of many, a great many new buildings have been constructed, especially in New York City, with these monstrosities on them, and have been accepted by the Building Department in all seriousness.

The fire wall should be introduced into all buildings where the public congregates in large numbers. Large department stores, which on certain days accommodate several thousand people per floor, are very dangerous places at present. A fire, or a panic without a fire, might cause a fearful tragedy. It is criminal for their owners to object to fire walls and offer as an excuse that they would obstruct the vista. Certain cities require fire walls in such buildings now as a property protection, and the vista is dispensed with without comment. The department stores of Philadelphia are so divided; John Wanamaker's new store there is divided by two such walls.

Churches, assembly halls and similar ground-floor buildings should have the floor fireproof and unpierced so that any fire occurring in the basement would not endanger the occupants of the main building.

Every school building should be divided by a fire wall providing a horizontal exit on each floor, so that the children will not have to be drilled to go down stairs in case of fire.

Hospitals where the inmates are bedridden, blind, lame, invalid, imbecile, or otherwise helpless, can be made safe by the introduction of the fire wall between wards, and in case of fire those who are bedridden can be wheeled on their beds through the doorways, and those who are up and about can walk through them.

Hotels and apartment buildings can so easily have a fire wall developed in them that it need only be referred to here in passing. Even the private residence where only a few people occupy a floor can be made safe in this way. The back stairway should be inclosed in a fireproof partition, and in case of a fire instead of everybody having to go down stairs through the smoke and flames, or having to jump from windows, the people on each floor have simply to pass through the fireproof door and go down stairs in safety. In large residences where there is a servants' quarters in connection with the back stairs, the building would be bisected and the people on either side of the wall would be able to carry their clothing and perhaps much household and personal property to safety.

The author advocates legislation, requiring (a) that architects and builders be prohibited from designing buildings which cannot be emptied within 3 minutes after a given signal; (b) that the municipal authorities be required to institute an exit test in each building to determine, before it is accepted, if it can be emptied of its occupants in 3 minutes. If it cannot pass this test it will not be accepted and must be altered until it can pass the test. (c) Afterward the proper authorities will be required to repeat the exit test from time to time, to see that the safe conditions originally established are maintained.

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A Sanitary Pail System for Brunswick, Ga., is to be installed for those portions of the city which are still without sewers. We are informed by J. Hunter Hopkins, Mayor of Brunswick, that the city has bought cans for the service, but has not as yet built the disposal station, nor worked out the method of handling the night soil. It is expected that the control of these operations will be in charge of the Sanitary Inspector of the city. The cost is to be a public charge.

Failure of Coffer at Lock and Dam No. 48, Ohio River

By J. C. Oakes*

At about 7 A. M., Monday, July 21, 1913, there occurred a failure of the main coffer-dam at lock and dam No. 48, Ohio River.† This coffer was built 20 ft. above low water, and at the time of the failure the river stage was 12.4 ft. above low water, or elevation 337.5 (Sandy Hook datum), the water surface within the coffer was at elevation 316.5, the head being 21 ft. (See accompanying cuts.)

The failure occurred at the passway in the lower arm. This passway was left to enable the contractor to pass floating plant in and out of the coffer at stages from 18 to 20 ft., and, as originally constructed, was 41 ft. wide with bottom 6 ft. below top of coffer, the area being closed by needles.

After work had been closed for the winter and the Government inspector withdrawn this passway was torn out

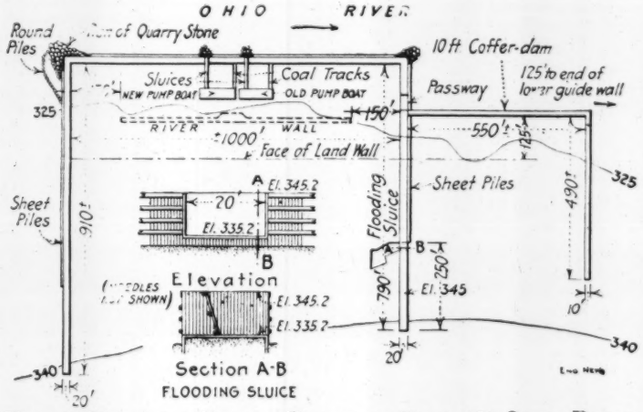

Fig. 1. Plan of Coffer-Dam for Dam 48, Ohio River

to remove some floating plant, and without the knowledge of this office was rebuilt with the top two feet lower than before. The top of the coffer, therefore, at this passway was only 12 ft. above low water, needles being used to close the opening when required. On the outside of the coffer was a line of Wakefield sheet piles 26 ft. long, with top of piles from 10 to 12 ft. above low water. These sheet piles extended about 10 ft. below the bottom of the coffer. Sand was backed against the coffer to a height of the tops of the sheet piles.

In order to hold the banking against the coffer on the inside the contractors had driven a line of sheet piles 60 or 65 ft. away from the coffer with tops of piles at about low-water elevation and fill had been placed sloping from the tops of these piles to the coffer at the elevation of the floor of the passway. This fill had been covered by gravel to prevent wash by seepage.

A small quantity of water, probably one cubic foot per second, has always collected behind the passway and flowed as a little stream into the excavated area, but as there were many of these little streams, some of them greater than this one, no fear of failure because of this seepage had been felt.

The Government inspectors lived on a quarterboat moored just below this passway, and crossed the coffer constantly at this point and no increase in the amount of seepage nor any movement of sand had been noticed.

The excavation was kept clear of water by three 15-in. centrifugal pumps on a pump boat, the latter resting on piles. The engineer of the pump boat noticed nothing unusual until 5:30 a.m., July 21, when he found the pumps were not holding the water surface at 315.5. He then changed his governors, increasing the speed of the engines, and finally cut out the governors entirely, allowing the engines to run full speed. In spite of this the water surface inside rose to 316.5 at 6:30 a.m. Shortly after this the men coming to work noticed a large leak near the passway, the alarm was given, and the pumps stopped at 6:45 a.m. At 7:01 a.m. there was evidently a blowout, as the water was seen to suck down outside of the sheet piles, the sheet piles were lifted out, then the coffer lifted, and the break was completed. Before the inclosure was filled about 250 ft. of coffer had been washed away.

Through the gap thus created there were drawn four

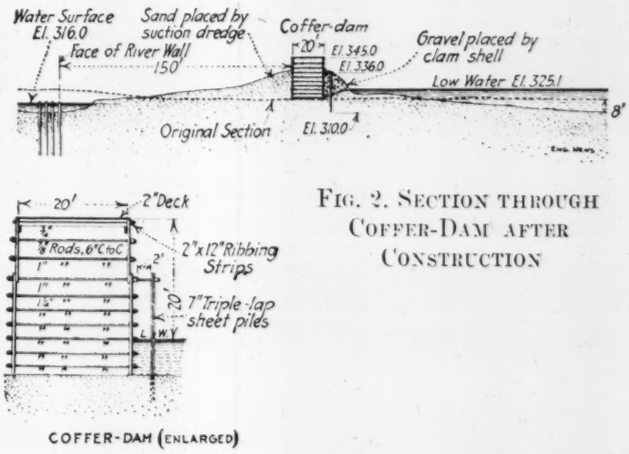

Fig. 2. Section through Coffer-Dam after Construction

loaded coal barges, a barge of lumber, and one of round piles. The coal barges were rolled over and over, and were a total wreck, the pile barge was broken up and the lumber barge was injured, but can be recovered and repaired. The pump boat was thrown off its pile foundation and somewhat injured, and four pile-drivers were submerged and probably injured to some extent, the amount of injury being unknown.

The contractor was about ready to place concrete for the river wall. The excavation was completed, all round piles and most of the sheet piles were driven and tracks on piles for derricks, cars, etc., were completed. A large amount of sand has been carried into the excavation, coal has been dumped about the heads of the piles, and undoubtedly some of the tracks have been injured. It is estimated that the immediate money loss to the contractor will amount to between $10,000 and $15,000, but the accident may cause a much greater loss due to the delay, which is estimated at a month or six weeks, which may prevent the completion of the work inside the coffer this season, and make necessary the unwatering again next year. It is hoped, however, that further investigation when the river falls will show less damage than is anticipated.

Several causes of the failure can be suggested, and it is probable that they all had a bearing; their relative importance, however, can only be guessed at. These probable causes are: (a) The weight of the passway coffer was probably about 1600 lb. per sq.ft. less than it would

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†This coffer-dam was described in an article in "Professional Memoirs, Corps of Engineers, U. S. A., and Engineer Department at Large," and reprinted in "Engineering News," Apr. 10, 1913, p. 702.—Ed. "Eng. News."

have been if it had been built to full height; (b) in rebuilding the passway, good connection between the old and new sheet piles and the sheeting of the coffer may not have been obtained; (c) the material used for filling the passway coffer when rebuilt may have contained a large proportion of silt, as the silt deposit in the coffer during the winter was very heavy; (d) the seepage probably gradually increased with the increasing head until, during the night, the little stream began to carry out sand from beneath the coffer and it also probably cut down into the inside banking until the limiting plane of saturation was reached, when the blowout occurred.

It is certain that there was a considerable increase of flow during the night from this and probably other seepage streams, and that at 6:30 a.m. there was a bad leak under the passway, and when the blowout took place the whole mass became suddenly fluid and lifted the sheet piles and the coffer.

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The Influence of Copper in Retarding the Corrosion of Steel Plate*

A series of tests was undertaken, as noted below, to establish the value of small amounts of copper in steel when exposed to natural corrosion under atmospheric conditions. To avoid uncertainty in comparing different heats of steel with and without copper and to keep identical conditions except copper contents, it was decided to copperize certain portions of each heat, leaving other portions in their original condition.

Three heats were used of the following analyses:

	Regular basic openhearth	Rephosphorized basic openhearth	Regular bessemer
Carbon	0.10	0.13	0.08
Manganese	0.34	0.45	0.46
Sulphur	0.034	0.036	0.079
Phosphorus	0.019	0.042	0.096

In pouring the openhearth heats several ingots were first poured without the introduction of copper and then sufficient copper was added to four ingots to obtain in two of them about 0.15% and in the other two about 0.25% copper (finished product). The bessemer heats were treated the same way except that only two ingots were copperized, since the average bessemer heat is too small to furnish six ingots of the size desired. Copper was added to the molds a little at a time as they were filling. Uniformity of resultant steel and copper content was demonstrated by many analyses of bars and finished sheets. It is a well established fact that copper diffuses through the molten steel and does not segregate on cooling.

Fifteen ingots (six from each of the openhearth heats and three from the bessemer) were carried through the usual mill operations, each bar as cut and each sheet as rolled being chalkmarked to prevent confusion. In the end each lot was carefully analyzed as a double check.

One ingot of each grade of openhearth was rolled into No. 16 gage and the other into No. 27 gage sheets, 30x96 in. One-half of each bessemer ingot was rolled into No. 16 and the other half into No. 27 gage. All grades were subjected to the same treatment by the same crews, and annealed in the same furnace at the same time. The finish was such as to conform with that of competitive sheets used in the test. From 24 to 36 sheets of each of the nine grades in both gages were sheared down to 24x96 in., giving a strip 6 in. wide from each sheet. From these, 2x4-in. test pieces were cut and stenciled with distinguishing marks. The 24x96-in. sheets were corrugated in the usual way and eight to twelve of each grade were shipped to each of three testing stations. One of these is located in the Pennsylvania coke regions where the air contains notable amounts of sulphuric acid and other fumes. Another station is located on the sea coast and the third is in a rural community where the air is pure and free from corrosive agents.

Service Tests

At each of these stations a wooden-skeleton building was erected, 40x80 ft. with a roof sloping at an angle of about 18°, entirely open and free to the passage of air on all four sides. The frameworks were uncovered until the sheets were put on, the purlins being 92 in. apart, thus allowing for a 2-in. hold on each end of the sheet. Sheets were arranged in panels, each grade being separated from the other by an open space; there were open spaces between courses so that the drip from one row did not run upon the next.

In addition to the nine grades mentioned there were purchased in the open market, for exposure at each test station under the same conditions, some No. 27-gage and 16-gage sheets of the following average analysis: Carbon 0.02, Manganese 0.03, Sulphur 0.034, Phosphorus 0.003, Copper 0.06 to 0.07.

All sheets were placed on the roofs during November, 1911, and were unprotected by paint or other coating. Within a short time the higher-copper steels showed an advantage. The panels which contained no copper were rough and gave evidence of pitting. The low-carbon and manganese materials with only 0.07% copper was not quite so rough and the pitting not quite so well developed, while the panels containing from 0.15 to 0.31% copper were fairly smooth with scarcely any pitting. The rust on non-copper steels was bright red and loosely adherent; the copper-bearing steels carried a dark brown closely adhering oxide. It was possible to distinguish them by their color at a considerable distance from the building.

The non-copper panels in the coke-region station were failing and falling off by June 27, 1912. All the copper steels were then still in excellent condition. By Dec. 7, the non-copper panels had entirely disappeared and the low-carbon manganese material had begun to drop off. All the high-copper-bearing panels were still intact. In March, 1913, the low-carbon manganese panels had entirely disappeared, while the high-copper sheets were still in place.

At the seashore station on Sept. 7, 1912, the panels were in about the same condition as those in the coke-region station some months earlier (a few weeks after these sheets had been placed a severe gale ripped off several panels). On Feb. 20, 1913, inspection of the rural district station showed that the copper-bearing materials had not failed, while the low-manganese was in poor condition and the non-copper sheets dropping.

Laboratory Tests

A selection of the 2x4-in. test pieces were carefully weighed on a chemical balance, mounted in wooden racks with free access to the weather and exposed at each sta-

*From a paper by D. M. Buck, chief chemist of the American Sheet & Tin Plate Co., read before the annual meeting of the American Chemical Society, Milwaukee, Wis., Mar. 25, 1913.

RELATIVE LOSSES ON 2X4-IN. STEEL TEST PIECES (EACH RESULT IS THE AVERAGE OF SIX PIECES)

Grade	Analysis Carb.	Mang.	Sul.	Phos.	Copper	Relative losses, 100 equals greatest Corrosion
Exposed in Coke Regions Nov. 21, 1911; Taken Down Aug. 14, 1912.						
Bessemer	0.05	0.44	0.082	0.101	0.23	39.09
Bessemer	0.05	0.44	0.075	0.099	0.34	39.61
Bessemer	0.07	0.46	0.069	0.095	0.33	41.57
Openhearth	0.10	0.46	0.035	0.043	0.17	42.09
Openhearth	0.07	0.47	0.038	0.043	0.25	42.22
Openhearth	0.06	0.33	0.035	0.018	0.25	43.27
Openhearth	0.06	0.35	0.036	0.018	0.16	43.92
Bessemer	0.08	0.45	0.070	0.094	0.21	44.05
Openhearth	0.14	0.46	0.038	0.043	0.27	46.67
Openhearth	0.13	0.44	0.035	0.042	0.18	46.67
Openhearth	0.10	0.35	0.033	0.019	0.23	47.32
Openhearth	0.10	0.34	0.035	0.020	0.16	48.36
Low carbon and low manganese material	0.02	0.03	0.036	0.003	0.07	50.19
Low carbon and low manganese material	0.03	0.03	0.034	0.003	0.06	53.20
Openhearth	0.13	0.45	0.035	0.042	0.00	74.64
Openhearth	0.09	0.47	0.037	0.043	0.00	78.16
Bessemer	0.08	0.46	0.070	0.098	0.00	91.64
Bessemer	0.05	0.45	0.076	0.100	0.00	96.86
Openhearth	0.10	0.34	0.034	0.019	0.00	98.82
Openhearth	0.06	0.35	0.033	0.018	0.00	100.00
Exposed at Sea Shore Nov. 28, 1911; Taken Down Sept. 7, 1912						
Bessemer	0.05	0.44	0.075	0.099	0.34	51.12
Bessemer	0.05	0.44	0.082	0.101	0.23	51.12
Openhearth	0.07	0.47	0.038	0.043	0.25	51.96
Openhearth	0.10	0.46	0.035	0.043	0.17	55.03
Openhearth	0.06	0.33	0.035	0.018	0.25	55.59
Bessemer	0.08	0.45	0.070	0.094	0.21	56.42
Bessemer	0.07	0.46	0.069	0.095	0.33	56.70
Openhearth	0.10	0.35	0.033	0.019	0.23	57.54
Openhearth	0.13	0.44	0.035	0.042	0.18	58.94
Openhearth	0.06	0.35	0.036	0.018	0.16	58.94
Openhearth	0.14	0.46	0.038	0.043	0.27	60.90
Openhearth	0.10	0.34	0.035	0.020	0.16	64.81
Low carbon and low manganese material	0.02	0.03	0.036	0.003	0.07	64.81
Low carbon and low manganese material	0.03	0.03	0.034	0.003	0.06	65.37
Openhearth	0.09	0.47	0.037	0.043	0.00	65.60
Openhearth	0.13	0.45	0.035	0.042	0.00	70.39
Openhearth	0.06	0.35	0.033	0.018	0.00	87.99
Openhearth	0.10	0.34	0.034	0.019	0.00	88.83
Bessemer	0.05	0.45	0.076	0.100	0.00	98.32
Bessemer	0.08	0.46	0.070	0.098	0.00	100.00
Exposed in Rural Community Dec. 3, 1911; Taken Down Nov. 16, 1912						
Openhearth	0.06	0.35	0.036	0.018	0.16	51.89
Bessemer	0.05	0.44	0.075	0.099	0.34	52.69
Openhearth	0.10	0.34	0.035	0.020	0.16	52.69
Bessemer	0.05	0.44	0.082	0.101	0.23	53.88
Bessemer	0.07	0.46	0.069	0.095	0.33	54.67
Openhearth	0.06	0.33	0.035	0.018	0.25	55.07
Openhearth	0.10	0.46	0.035	0.043	0.17	55.27
Openhearth	0.13	0.44	0.035	0.042	0.18	55.27
Openhearth	0.07	0.47	0.038	0.043	0.25	55.67
Openhearth	0.10	0.35	0.033	0.019	0.23	55.87
Bessemer	0.08	0.45	0.070	0.094	0.21	57.26
Low carbon and low manganese material	0.03	0.03	0.034	0.003	0.06	57.46
Openhearth	0.14	0.46	0.038	0.043	0.27	58.25
Low carbon and low manganese material	0.02	0.03	0.036	0.003	0.07	64.42
Openhearth	0.13	0.45	0.035	0.042	0.00	68.79
Openhearth	0.09	0.47	0.037	0.043	0.00	71.77
Openhearth	0.10	0.34	0.034	0.019	0.00	76.54
Bessemer	0.08	0.46	0.070	0.098	0.00	90.26
Openhearth	0.06	0.35	0.033	0.018	0.00	90.66
Bessemer	0.05	0.45	0.076	0.100	0.00	100.00

tion. After a considerable time had elapsed and certain pieces had rusted entirely through, they were taken down and reweighed after first removing all rust by a solution of ammonium citrate which takes off the oxide without attacking the underlying metal.

The accompanying table shows the results. The figures are arranged to show the relative values on the basis of 100 points for greatest corrosion at each station with the others in their relative values. The results on these weighed test pieces check up very closely with the results on full-size sheets.

Accelerated acid tests were made on a selection of 2x4-in. test pieces. It was found that the copper-bearing, openhearth and bessemer steels resisted the acid from 50 to 100 times as well as the non-copper steels; within the limits of the copper contents of the steels used, the resistance to acid was directly proportional to the amount of copper present. In this regard, the acid tests differed from the actual weather tests wherein the copper-bearing steels resist atmospheric disintegration from 1½ to 2 times as well as normal steels without copper and there was little or no difference in the average between a copper content of 0.15 and 0.30%.

While a certain relation is established between the accelerated acid test and natural corrosion when comparing copper-bearing and non-copper steels or irons, yet many instances have been noted of great inconsistencies. It is the experimenter's opinion that the accelerated acid test is untrustworthy and misleading. Should copper-bearing steels be desired the presence or absence of that element should be determined by chemical analysis, according to his ideas.

It is well known that copper is electro-negative to iron and when placed in contact with iron it will stimulate corrosion of the latter. The reverse action when copper is alloyed with iron may be due to the alloy taking the noncorrosive properties of copper. The alloy of copper and iron may be less electro-positive to the first film of rust formed than is non-copper steel, with a consequent decrease in difference of potential lessening the corrosion. It has been suggested by W. H. Walker that the copper prevents the oxides of manganese and iron, which may be present, from coming out of solid solutions as the melt cools and hence, although the oxides are still present, they are uniformly dissolved and not segregated between the iron crystals as is normally the case. Sufficient work has not been done to form any definite conclusions about internal reactions. The author has intended only to publish results which seem to prove that a small copper content (approximately 0.2%) materially increases the life of steel sheets subjected to atmospheric corrosion.

NOTES FROM THE ENGINEERING SCHOOLS

At the East Pittsburgh plant of the Westinghouse Electric & Manufacturing Co., 28 teachers of electrical enginering in 26 colleges and schools of the country are spending their vacations in practical work. This summer course has been gradually built up so at the present time there are four objects laid out; (1) to obtain a working knowledge of methods of commercial electrical engineering, (2) to secure the opinions of successful engineers and manufacturers as to the qualifications of men needed in industrial work, (3) an exchange of ideas among the teachers themselves, and (4) a closer study of the industries of the Pittsburgh district. Many of these teachers are heads of electrical engineering departments and the average length of time spent in teaching is 5¼ years. The men are employed on the regular payroll and are under shop hours and rules. They receive compensation at the rate of 22 cents per hour, the rate for second-year apprentices.

At the close of evening work hours these men usually get a hasty lunch and proceed on trips of inspection guided by shop foremen and superintendents. In the evening lectures are given by company engineers, managers and foremen and by members of their own number.

IOWA STATE COLLEGE—The Iowa legislature has appropriated $65,000 for the erection, at the Iowa State College, Ames, Iowa, of a new transportation building, for which plans and specifications are prepared. It will include a laboratory for testing locomotives and electric

cars. Another laboratory will be provided for the testing of brakes and signal equipment. In addition to railway transportation, highway engineering will be given prominent consideration.

The law which constituted the State College the State Highway Commission, has been repealed by the legislature, and a new highway commission created. The new commission now consists of three men, the chairman of which is the Dean of Engineering at the State College, the other two are appointed by the governor. The headquarters of the highway commission are fixed by law at the college, but it is not in any way a part of the same. The commission is organizing a corps of about 30 engineers and office employees. (See ENG. NEWS, July 17 1913, p. 131, for synopsis of new law, etc.)

Fire Destroyed the Coasting Steamer "Bangor," on July 23, while lying at the Atlantic Works, East Boston, Mass. She was owned by the Eastern Steamship Corporation.

A Butting Collision between two ore trains on the Duluth & Iron Range R.R., near Colby, Minn., on July 22, caused the deaths of two men. The line is double-tracked at this point and press dispatches state that one of the trains had been running on the wrong track for five miles.

A 30-In. Water Main Burst in Philadelphia, Penn., July 20, causing a local water famine of four hours' duration. A hole about 20 ft. diameter and 15 ft. deep was washed in the street.

The Throttle Failed to Operate on a switch engine of the Baltimore & Ohio R.R., at Cincinnati, Ohio, July 20, causing the engine to collide with a string of freight cars. The engineer and the fireman jumped, the former sustaining slight injuries.

Gatun Lake was 48.25 ft. above sea level when the sluicegates in the Gatun dam spillway were closed on June 27. On July 12, it had risen 3.65 ft. and stood at 51.9 ft. The rainfall from July 1 to July 12 varied from 5.25 in. at Gatun to 3.28 in. at Alhajuela.

Superheaters are to be applied to all the locomotives on the New York, New Haven & Hartford R.R. The estimated expenditure is nearly $1,000,000, and the locomotives will be equipped at the rate of five per month. The work will be done at the Readville, Mass., shop, and about 350 locomotives in all will be altered.

Fire in a Chemical Plant, at North Weymouth, Mass., owned by the American Agricultural Chemical Co., on July 22, did large damage. Press dispatches state that the fire was caused by sparks from a pile-driver engine reaching a quantity of saltpeter being unloaded from lighters. Sparks from the resulting explosions set fire to the main building in several places.

A Wooden Prison "Cage" Burned, July 21, causing the death of 35 negro convicts, at the Oakley state convict farm near Jackson, Miss. The farm is not equipped with any form of fire protection. Neither of the two night watchmen saw the fire until it was beyond control. The structure was built 10 years ago, of lumber taken from the old penitentiary building in Jackson, and had but one exit.

A Railway Embankment Slid Into a Lake near Piscotasing Station, on the Canadian Pacific Ry., July 10, drowning two sectionmen who were sitting upon the bank at the time. A bridge which was near the scene of the accident was carried away by the slide, and the trains dependent upon it were held up for 12 hr.

A Sheet-Iron Cornice, about 25 ft. in length, was loosened from its fastenings on the rear of a frame building at South Second St. and Marcy Ave., Brooklyn, N. Y., on July 22; and crushed down through the roof of adjacent one-story buildings. Fortunately no one was in any one of the houses on which the mass fell, but the wreckage was very great, the loss being estimated at $10,000.

A Hudson Bay Supply Ship, the "Biothic," was wrecked on the northwest coast of Newfoundland, on July 21, near the western entrance of the Straits of Belle Isle. The vessel was of 1140 tons burden and was loaded with supplies, fuel, etc., for use by the survey and construction parties at work on the Hudson Bay Ry.

A Car Turned Over on the Erie R.R., near West Nutley, N. J., July 27, killing four laborers who were unloading dirt from it to strengthen an embankment. The dirt on the left side of the car (the center car of a seven-car work train) had been removed, when the train rounded a sharp curve at a speed of about 5 mi. per hr. The weight of the dirt on the right side, combined with the pull on the same side due to centrifugal force, caused the car to turn over, throwing the workmen down a 15-ft. embankment.

Ventilation Problems are to be studied experimentally during the next four years, under the direction of the following commission: Prof. C.-E. A. Winslow, of the College of the City of New York; Prof. F. S. Lee, of the College of Physicians and Surgeons, Columbia University; Prof. E. L. Thorndike, Teachers College, Columbia University; Prof. E. B. Phelps, Massachusetts Institute of Technology; Dr. James Alexander Miller, New York City, and D. D. Kimball, of Richard D. Kimball Co., 15 West 38th St., New York City. A sum of $50,000 has been appropriated for the study, from a gift made by Mrs. Elizabeth Milbank Anderson to the New York Association for Improving the Condition of the Poor.

The Flood-Protection Investigation at Columbus, Ohio, is making good progress. The field work is drawing to a close, and the office work consists in laying out different projects for the control of the river through the west side of the city. Four large storage reservoirs have been selected in the upper watershed of the Olentangy and Scioto rivers. Their storage areas have been computed and their cost determined, and the modifying influence of this amount of storage is being studied. The investigation is in the hands of a board of engineers, composed of Messrs. John W. Alvord, of Chicago, Daniel W. Mead, of Madison, Wis., and S. M. Woodward, of the University of Iowa. A similar investigation is being made for Dayton, Ohio, but this has not advanced so far as the work in relation to Columbus.

New York City is now the greatest seaport in the world, measured by the total value of its exports and imports. For the fiscal year ending June 30, 1913, according to figures collected by the Merchants Association of New York City, the port of New York did a foreign business of nearly two billion dollars. The figures for New York and nine other great ports are given as follows:

	Total trade
1. New York	$1,973,981,693
(Fiscal year ending June 30, 1913.)	
2. London	1,791,857,641
3. Hamburg	1,674,187,176
4. Liverpool	1,637,280,476
5. Antwerp	1,121,654,799
6. Marseilles	678,431,300
7. Havre	531,096,600
8. Bremen	501,146,540
9. Buenos Aires	479,536,241
10. Calcutta	410,128,830

Notwithstanding the current idea that trade has been dull during the past six months, the figures show that the business of the port of New York increased about $200,000,000 last year over the record of the year preceding.

The Structural Steel Reinforcement of a 125-Ft. Concrete Arch of the West 7th St. Viaduct Fort Worth, Tex., fell during erection on July 11, carrying down with it five men who were at work there. One of these men had his leg broken and others were slightly injured. The reinforcement fortunately did not fall far as the distance to the bed of the stream was not very great and it was not seriously injured. With the exception of a few of the members of the crown, which were bent by striking the bed of the river and which will have to be replaced with new material, the steelwork is uninjured. Work of reërecting the span is now under way.

This structure is one of several now being built in Fort Worth and was described together with the others in "Engineering News," Dec. 12, 1912, p. 1097. The arch in question is 125-ft. span over the Trinity River. The reinforcement for the concrete arch consists of three structural steel ribs having ball and socket hinges at the springing line and at the crown. The three ribs are braced together at 12-ft. 6-in. intervals with transverse structural steel girders. There is also a double system of lateral rods, one at the top and one at the bottom chord of the rib. The ribs were erected on light falsework, the rods were put in and tightened up, the falsework was then removed and the riveting of the splices and connection of the transverse struts were started.

The accident was caused by the fact that when the riveting of the transverse strut connections was under way the erector removed quite a number of the lateral rods where they interfered somewhat with driving the rivets. We understand that these rods were removed in spite of the protest of the engineers on the work, Brenneke & Fay, of St. Louis, Mo. A stiff wind was blowing down the valley at right angles to the span at the time of the accident and the structure being in unstable equilibrium after the rods were removed, with nothing but the stiffness of the connections to hold it in place, was not sufficiently strong to withstand the wind pressure.

A Decision on Going Value of the property of a public-service corporation, appreciation of plant due to improved paving over street mains, etc., has recently been rendered by the Appellate Division of the New York Supreme Court in the case of the Kings County Lighting Co. vs. the Public Service Commission for the First District. This is an action to review a determination of the Commission fixing the rate for gas as noted in "Engineering News," of Nov. 16, 1911. The company challenged the valuation of the Commission and the court has brought forward the following points.

The company claimed $600,000 as the value of its established business. The commission in its first report held that this included capitalized profits of very good street-lighting contracts, which might be allowed in a case for taking over the property as a whole, but it would not allow it for making rates. The court holds it can find no logical difference between allowing the one value under these two conditions, and intimates that if admissible for one is for the other.

The company claims that it should be allowed $200,000 increase in value of its gas mains due to the present paving in the street over them compared with the paving which existed when the mains were put down. The commission did not allow this but the court holds that it is the fact of present value, and not the means by which the appreciation in value has been attained, which is to be considered. It holds that the increased value of the mains under improved paving is a necessary element of reproduction value under present conditions, which may be taken in figuring actual present value. The court admits that the appreciation is unearned increment, but holds that the present law must include it in its definition of property capable of ownership and entitled to protection.

The company held that it should earn 100% on the reproduction value of its property in contrast with the allowance of the commission for earning a fair rate on the value reproduced less depreciation accrued to date. The court holds that it is the nearest approach to value of the tangible property at the present time which is to be considered, and that a deduction must be made to represent the amount required to replace at the end of useful service apparatus still in use, but in process of wearing out.

One of the notable tenets of the commission, as reported in "Engineering News," of Aug. 31, and Nov. 16, 1911, was the idea that annual increse in value of the company's land was to be included as income and assets in offsetting the depreciation. The court holds that the land is used for the business of the company and as long as it is so held any increase in value cannot be considered as income or available for the payment of debts, or dividends.

The court fixed no rate as being suitable, but left it to the commission to fix one on the principles stated in the opinion.

PERSONALS

Mr. R. A. Cook, Engineer of Track Elevation of the Chicago & Alton R.R., at Joliet, Ill., has been appointed Valuation Engineer of the railway, with office at Chicago, Ill.

Mr. L. R. Palmer, of Pittsburgh, Penn., has been appointed Chief of the Bureau of Factory Inspection of the new Department of Labor and Industry of Pennsylvania. The salary is $5000 per annum.

Mr. W. N. Mitchell, recently at the head of the railway department of the International Correspondence Schools, at Chicago, Ill., is now Fuel Supervisor of the Chicago Great Western Ry., at Chicago.

Mr. J. H. O'Neil, Division Superintendent of the Great Northern Ry., at Everett, Wash., has been promoted to Assistant General Superintendent of the Western district, with headquarters at Spokane, Wash.

Mr. M. Nicholson, Division Superintendent of the Great Northern Ry., at Melrose, Minn., has been promoted to be Assistant General Superintendent of the Central district, with headquarters at Great Falls, Mont.

Mr. J. C. McClure, M. Am. Soc. C. E., recently Assistant General Manager of the Southern Pacific R.R. of Mexico and the Arizona Eastern R.R., at Tucson, Ariz., has been appointed Assistant to the President of both these railways.

Maj. James P. Jervey, M. Am. Soc. C. E., Corps of Engineers, U. S. A., Resident Engineer in charge of masonry construction at the Gatun Locks, Panama Canal, since 1902, has been transferred to the Chief of Engineer's office at Washington, D. C.

Mr. C. W. Jones, recently Vice-President and General Manager of the Chicago, Rock Island & Gulf Ry., has been elected President and General Manager, with office at El Reno, Okla., succeeding Mr. H. U. Mudge, who becomes Chairman of the Board of Directors.

Mr. J. H. Nuell, recently Engineer of Maintenance-of-way of the New York, Ontario & Western Ry., has been appointed Chief Engineer. Mr. Nuell was formerly Assistant Engineer at Norwich, N. Y., and was promoted to be Principal Assistant Engineer in September, 1911. He was made Engineer of Maintenance-of-way in January, 1912.

Mr. L. H. Long, Assistant General Manager of the Southern Pacific R.R. of Mexico and the Arizona Eastern R.R. has been promoted to be Vice-President, with headquarters at Tucson, Ariz. Mr. Long is an engineer by training and experience and his recent position under the Hine system of organization corresponded to that of Chief Engineer.

Mr. H. D. Batchellor, recently Assistant to the General Superintendent of the Baltimore & Ohio Southwestern R.R. and the Cincinnati, Hamilton & Dayton Ry., at Cincinnati, Ohio, has been appointed District Engineer of Maintenance-of-way, with headquarters at Cincinnati. Mr. Batchellor was Division Engineer of Maintenance-of-way, at Seymour, Ind., until January, 1913.

Mr. F. S. Elliott, recently Assistant General Superintendent of the Western district of the Great Northern Ry., at Spokane, Wash., has been promoted to be General Superintendent of the Central district with office at Great Falls, Mont., succeeding Mr. C. O. Jenks, who has been transferred to Superior, Wis., to succeed the late J. H. Taylor, General Superintendent of the Lake district. Mr. Eliott's railway work began in 1882 as a telegraph operator with the Chicago, Rock Island & Pacific Ry.

Mr. Samuel Porcher, Assistant Purchasing Agent of the Pennsylvania R.R. has been promoted to be Purchasing Agent, succeeding the late Daniel S. Newhall. Mr. Porcher was born in South Carolina and educated at the University of Virginia. He entered the service of the Pennsylvania R.R. in 1882 as an apprentice at the Altoona, Penn., shops. He remained at Altoona until 1888, when he was appointed Assistant Engineer of Motive Power of the United R.R. of New Jersey division of the Pennsylvania R.R., at Jersey City, N. J. He has been Assistant Purchasing Agent since 1894.

Mr. George M. Wells, M. Am. Soc. C. E., formerly Office Engineer, Atlantic division, Panama Canal, has been appointed Resident Engineer in charge of all the municipal engineering work of the Atlantic, Central and Fifth divisions of the Panama Canal, to be known as the Division of Municipal Engineering under the permanent organization which will have charge of the operation of the completed canal. Mr. Wells is one of the old timers, being one of the first American engineers to go to the Isthmus after the work was taken over from the French in 1904. He studied at the Michigan College of Mines and in 1901 was appointed Junior Engineer in the United States Engineer Department. He has served in various capacities on the Panama Canal from the first survey work through the fever-infected swamps to a responsible position at Gatun under Lieut.-Col. W. L. Sibert, Division Engineer of the Atlantic division.

Mr. John C. Sesser, Assoc. M. Am. Soc. C. E., Assistant Engineer of Maintenance-of-way of the Great Northern Ry., at St. Paul, Minn., has been appointed Superintendent of the Kalispell division, with headquarters at Whitefish, Mont., succeeding Mr. W. R. Smith, transferred to the Cascade division at Everett, Wash. Mr. Sesser was born at St. Joseph, Mich., and graduated from Lehigh University with the degree of Civil Engineer in 1896. After a few months' experience as draftsman with the Allentown Rolling Mill Co., Allentown, Penn., he entered the railway service as an instrumentman and was afterward Resident Engineer on the Chicago & Northwestern Ry. After some months' experience as Resident Engineer on the Union Pacific R.R., in December, 1899, he went to Hawaii, where he spent a year as Chief Engineer of the Olas Plantation, Hilo. In 1901 and 1902 he was Resident Engineer with the Chicago, Milwaukee & St. Paul Ry., and for a short time Chief Engineer of the Iowa & St. Louis Ry. From 1903 to 1907 he was Resident Engineer and Engineer of Maintenance-of-way of the Missouri district of the Chicago, Burlington & Quincy R.R. In 1907 he went into contracting as Vice-President of the W. K. Kenly Co. and as Contracting Engineer with the Walsh Construction Co., of Davenport, Iowa. Since 1909 he has been Assistant Engineer of Maintenance-of-way of the Great Northern Ry.

Mr. Howard Elliott, President of the Northern Pacific Ry., has been elected President of the New York, New Haven & Hartford R.R., to succeed Mr. Charles S. Mellen, resigned

as noted in our issue of last week. It is stated in the announcement that changes in the by-laws will be made which will make Mr. Elliott Chairman of the Board of Directors and thus executive head not only of the New Haven but of the Boston & Maine and various other controlled railways, trolley and steamship lines, each of which will have a President in charge of operation, who will be under the authority of the Chairman. Mr. Elliott was born in New York City on Dec. 6, 1860. He graduated from the Lawrence Scientific School, Harvard University, with the degree of Civil Engineer in 1881. His only real engineering experience, however, was a few months as rodman with the engineering corps of the Chicago, Burlington & Quincy R.R. In the fall of 1881 he accepted a position as a clerk in the President's office of the St. Louis, Keokuk & Northwestern R.R., and thereafter his experience was confined to the financial, traffic, operating and executive departments of the railway service. He was successively Assistant Auditor, Assistant Treasurer, General Freight and Passenger Agent and General Manager of the Chicago, Burlington & Kansas City Ry., the St. Louis, Keokuk & Northwestern R.R., the Hannibal & St. Joseph R.R. and the Kansas City, St. Joseph & Council Bluffs R.R. In 1902 he was elected Second Vice-President of the Chicago, Burlington & Quincy R.R. On Oct. 21, 1903, he was elected President of the Northern Pacific Ry. to succeed Mr. Charles S. Mellen, who had resigned to become President of the New York, New Haven & Hartford R.R. Mr. Elliott is to succeed Mr. Mellen as President of the New Haven system Sept. 1.

Mr. Otto Lemberger, who was obliged to abandon his engineering and surveying work in Mexico about a year ago because of the political disturbances in that country, has returned to his native country of Austria. While in New York he became interested in stereophotographic surveying and will be remembered by the readers of "Engineering News" as the author of the article on this subject in our issue of Mar. 27, 1913. Since returning to Austria he has had an opportunity of continuing his studies in this work. He writes us under date of July 16, from Vienna, that he is now studying under Capt. von Orel, the inventor of the stereoautograph, who has resigned from the Austrian Military Geographical Institute and established an institute in Vienna for instruction in stereophotographic surveying. Mr. Lemberger writes: "The apparatus has been very much improved and looks entirely different from that which I described. The present model of the stereoautograph covers an area of 72 sq.ft. and looks rather like a machine than a scientific instrument, with its many complicated mechanical devices. All the different operations as adjusting the plates, setting the base line to the desired scale, turning off the allowance for inclination and convergence are now done very quickly. The price of these new instruments is about $6000.

"Capt. von Orel's institute was founded for the reason that the handling of the very delicate apparatus can be done successfully only by men trained in its use and who can devote their whole time to this kind of surveying. It is not intended to have this new method of surveying brought into discredit by inexperienced users of the instruments. The stereoautograph is, therefore, not for sale to private parties; except in the case of governments, and they must sign an agreement that they will not use it to do private work. Otherwise the Stereographic Bureau, as the institute is called, sends out its own engineers to do the necessary field work and makes the topographical maps. The apparatus is already so improved that it is possible to work from photographic plates taken at equally inclined angles to the base line, as well as for convergent angles."

OBITUARY

Channing Whitaker, a mill architect and engineer, of Lowell, Mass., died at his home in Tyngsborough, Mass., July 23, aged 69 years. He served while a young man as soldier in the Union Army during the Civil War. After the war he entered the Massachusetts Institute of Technology, where he graduated with the second class of that institution in 1869, in mechanical engineering. From 1877 to 1887 he was the head of the department of mechanical engineering at the Institute.

Zephaniah Hopper, remembered by hundreds of prominent men, including many engineers and railway officers, as an instructor in mathematics at the Central High School, Philadelphia, Penn., died July 21, at the age of 89 years. He had been a teacher for 71 years and instructor at the Central High School for 59 years. In all these years he is said to have lost but 10 days of his school work. He was born in Philadelphia and graduated with the first class of the Central High School, at the time A. D. Bache, M. Am. Soc. C. E., afterward Superintendent of the United States Coast Survey, was Principal. It is estimated that during his term of service at the Central High School he had taught more than 25,000 boys the rudiments of higher mathematics. Outside of his school work he was widely known as a botanist.

Anthony N. Brady, until recently President of the New York Edison Co., famous as a promoter of electric railway and light companies and as a financier, died in London, England, July 22. He was born in France in 1843, and came to America with his parents when a small child. He never attended a school after he was 13 years old. At 21 years he opened a tea store in Albany, N. Y., and with others soon monopolized the tea trade of Albany and Troy. Later he became the owner of large granite quarries and with others purchased the gas plants of Albany, Troy and Chicago. He early entered the street-railway field and was one of the originators, if not the originator, of the consolidation and capitalization schemes which netted him and his coöperators many millions of dollars. He planned the notorious consolidation of the surface street railways of New York City, and promoted similar consolidations in Washington, Philadelphia and other cities. He was an officer or director of some 60 corporations and left a fortune estimated at $50,000,000.

ENGINEERING SOCIETIES

COMING MEETINGS

INTERNATIONAL GEOLOGICAL CONGRESS.
Aug. 7-14. Twelfth annual session at Toronto, Ont. General Secy., R. W. Brock, Director of Geological Survey of Canada, Ottawa, Ont.

THE TRAVELING ENGINEERS' ASSOCIATION.
Aug. 12-16. Annual convention at Chicago, Ill. Secy., W. O. Thompson, New York Central Car Shops, East Buffalo, N. Y.

AMERICAN PEAT SOCIETY.
Aug. 18-20. Joint annual convention of the Canadian and American Peat Societies, at Montreal, Can. Secy., Julius Bordollo, Kingsbridge, New York City.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Aug. 18-21. Annual convention at Butte, Mont. Secy., Bradley Stoughton, 29 West 39th St., New York City.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Aug. 19-22. Annual Convention at Watertown, N. Y. Secy., Clarence R. George, Houston, Tex.

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Sept. 1-4. Annual convention at Cleveland, Ohio. Secy., J. D. Farasey, 37th St., Cleveland, Ohio.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Sept. 1-6. Annual convention at New York City. Secy., Jas. McFall, Roanoke, Va.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 8-12. Annual convention at Chicago, Ill. Secy., L. C. Ryan, Chicago and North Western Ry., Stirling, Ill.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 9-11. The 24th general meeting at Denver. Secy., Prof. Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 10-12. Annual convention at Philadelphia, Penn. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 9-13. Annual meeting at Colorado Springs, Colo. Secy., S. M. Guma, 755 Boyleston St., Boston.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 9-12. Annual convention at Ottawa, Ont. Secy., A. P. Dane, Reading, Mass.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 22-26. Annual Convention at Pittsburgh, Penn. Secy., I. D. Israel, 29 W. 39th St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 22-24. Annual meeting at Pitsburgh, Penn. Chairman, H. M. Wilson, 40th and Butler Sts., Pittsburgh, Penn.

American Concrete Institute—The Board of Direction of the National Association of Cement Users, an organization formed in 1905, has issued the following bulletin:

Pursuant to instructions of the Kansas City and Pittsburgh conventions to effect a change of name of this organization, the Board of Direction desires to announce that the charter of the National Association of Cement Users, issued under the laws of the District of Columbia, was amended on July 2, 1913, so that the name of this Association is now changed to American Concrete Institute.

The first annual convention of the American Concrete Institute, which is the tenth convention of this organization, will be held in Chicago, Feb. 16-20, 1914.